

**MAINE DEPARTMENT OF TRANSPORTATION  
BRIDGE PROGRAM  
GEOTECHNICAL SECTION  
AUGUSTA, MAINE**

**GEOTECHNICAL DESIGN REPORT**

*For the Replacement of:*

**GOSLINE BRIDGE  
HIGH STREET OVER COLD STREAM  
WEST GARDINER, MAINE**

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Kennebec County  
WIN 23090.00

Soils Report 2021-40  
Bridge No. 2321

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## **1.0 INTRODUCTION**

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Gosline Bridge which carries High Street over Cold Stream in West Gardiner, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, geotechnical design parameters, and construction recommendations for the new box culvert.

The existing Gosline Bridge was constructed in 1959. The structure consists of a single 17-foot span, steel structural plate arch supported on timber grillage. According to the 2019 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the bridge is structurally deficient. The culvert is rated a 3 with minor rust staining, surface rust, and some rust flaking.

The proposed replacement structure is an 18-foot span and 7-foot rise, 74-foot long, precast concrete box culvert. The box culvert shall have 1-foot tall precast headwalls and 2-foot deep toe walls at the inlet and outlet. The upstream and downstream ends of the culvert will be slope-tapered at 1.75H:1V (horizontal:vertical). The box culvert invert will be embedded 2 feet into the streambed. To provide a stable subgrade for the installation of the box culvert, a 2-foot-thick bed of crushed stone wrapped in geotextile and reinforced with geogrid is recommended.

The new box culvert will be located on horizontal and vertical alignments that will approximately match existing alignments. The existing bridge will be closed during construction and traffic detoured onto state and local roads.

## **2.0 GEOLOGIC SETTING**

Gosline Bridge carries High Street over Cold Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Gardiner Quadrangle, Open-File No. 09-8 (2009), indicates the surficial soils in the vicinity of the bridge project consist of glaciomarine deposits of the Presumpscot Formation. These deposits generally consist of clay and silt that washed out of the Lake Wisconsin Glacier and accumulated on the ocean floor when the relative sea level was higher than at present.

The MGS Bedrock Geology of Maine (1985) maps the bedrock in the vicinity of the project as intrusive Syenite, Granofels of the Vassalboro Formation, and Schist of the Waterville Formation. The bedrock cored in the test borings drilled at the site consisted of Granofels (Gneiss) the Vassalboro Formation.

### **3.0 SUBSURFACE INVESTIGATION**

Two test borings explored subsurface conditions at the project location. Boring BB-WGCS-101 was drilled behind the existing culvert at the southwest corner. Boring BB-WGCS-102 was drilled behind the existing culvert at the northeast corner. Both borings terminated in bedrock cores. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The test borings were drilled on October 29 and 30, 2019 by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

The borings were performed by using a combination of solid stem auger, cased wash boring, and rock coring techniques. The borings were completed by backfilling and compacting the borehole with drill cuttings. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in June 2019. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.886 to the raw field N-values. This hammer efficiency factor (0.886) and both the raw field N-value and corrected N-value ( $N_{60}$ ) are shown on the boring logs.

Bedrock was cored in the borings using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the cores calculated. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field-testing requirements, and reviewed the field logs for accuracy. A MaineDOT NETTCP Certified Subsurface Inspector logged the subsurface conditions encountered in the borings. The borings were located in the field using taped measurements at the completion of the drilling program and then located by MaineDOT Survey.

### **4.0 LABORATORY TESTING**

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of one standard grain size analysis with natural water content, three grain size analyses with hydrometer and natural water content, and three Atterberg limit tests. The results of soil tests are included as Appendix C – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

## **5.0 SUBSURFACE CONDITIONS**

Subsurface conditions encountered in the test borings generally consisted of Fill, Glaciomarine Deposits, Marine Sand, and metamorphic Bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

### **5.1 Fill**

A layer of variable fill was encountered in borings BB-WGCS-101 and BB-WGCS-102. The thickness encountered was approximately 11 to 15 feet. The fill materials encountered consisted of:

- Brown, moist, SAND, little to some silt, little to some gravel;
- Dark brown, medium dense, Silty, SAND, some organics, some timber cribbing;
- Boulders.

Corrected SPT N-values in the fill layer ranged from 9 to 22 bpf, indicating the fill is loose to medium dense in consistency. One grain size analysis conducted on a sample of the fill indicated the material is classified as A-2-4 under the AASHTO Soil Classification System and SM under the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 12 percent.

Boulders and timber were encountered at the approximate invert elevation of the existing pipe arch.

### **5.2 Glaciomarine Deposits**

A layer of Glaciomarine Deposits was encountered in the test borings below the fill. The encountered thickness was approximately 5 to 8.5 feet. The deposit generally consisted of:

- Grey, wet, Clayey SILT, trace fine sand;
- Grey, wet, SILT, some clay, trace gravel, trace fine sand; and
- Grey, wet, Silty CLAY, trace sand.

One corrected SPT N-value in the upper glaciomarine deposit ranged was 9 bpf, indicating the crust of the deposit is medium stiff.

In-situ vane shear tests were conducted with Geonor rectangular vanes in the Glaciomarine deposits. A 55 x 110 mm vane was used. Six (6) successful vane shear tests conducted within the glaciomarine deposit showed measured undisturbed undrained shear strengths ranging from approximately 491 to 1205 psf, indicating that the deposit is medium soft to stiff in consistency. The remolded shear strengths at the test intervals ranged from approximately 89 to 268 psf. Based on the ratio of peak to remolded shear strength at all test intervals, the silt-clay deposit has a sensitivity ranging from 2.7 to 6 and is classified as moderately sensitive to sensitive (Bjerrum, 1954).

Atterberg limits tests were conducted on three samples of the Glaciomarine deposit and are summarized below:

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-WGCS-101, 2D	Clayey SILT	35.1	38	22	16	0.8
BB-WGCS-102, 3D	SILT, some clay	40.6	35	21	14	1.4
BB-WGCS-102, 4D	Silty CLAY	42.7	29	22	7	3.0

The plasticity indices of the samples indicate that the soils have slight plasticity (Burmister, 1949). The natural water contents of the tested samples ranged from approximately 35 to 43 percent and liquid limits ranged from 29 to 38. The liquidity indices range from 0.8 to 3.0. Interpretation of these results indicates that the soils with liquidity indices of 1 or less are preconsolidated, while those with liquidity indices in excess of 1 are on the verge of being a viscous liquid as the natural water content exceeds the liquid limit. Soils with liquidity indices in excess of 1 have a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are also indicative of soils that are unconsolidated and are commonly referred to as “quick.”

Three grain size analyses resulted in the material being classified as A-4 or A-6 under the AASHTO Soil Classification System and CL under the USCS. The natural water contents of the samples tested ranged from approximately 35 to 43 percent.

### 5.3 Marine Sand

Marine Sands were encountered beneath the Glaciomarine Deposits in the borings. The deposit encountered generally consisted of grey, wet, Silty, fine SAND, some gravel. One corrected SPT N-value in the deposit of 46 bpf was recorded, indicating the layer is dense in consistency.

## 5.4 Bedrock

Bedrock was encountered and cored in both borings. The following table summarizes approximate depth to bedrock, corresponding approximate top of bedrock elevation, and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (%), (R1, R2)
BB-WGCS-101	3+46.4	8.3 Rt	22.0	151.4	92, 97
BB-WGCS-102	3+74.2	7.6 Lt	21.6	150.1	85, 95

The bedrock at the site is identified as purplish to greenish-white, banded, fine-grained, hard to very hard, fresh, weakly foliated to massive Granofels (Gneiss) of the Vasssalboro Formation. The RQD of the bedrock ranged from 85 to 97 percent corresponding to a rock quality of good to excellent. Detailed bedrock descriptions and RQD are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. Rock core photographs are included in Appendix B – Rock Core Photographs.

## 5.5 Groundwater

Groundwater was measured at depths ranging from 2.5 to 12 feet below the roadway surface upon completion of the borings. Note that water was introduced into the boreholes during drilling operations and the measured levels may not represent stabilized groundwater elevations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

## 6.0 FOUNDATION ALTERNATIVES

A precast concrete box culvert was the only bridge replacement alternative considered for this project. A precast concrete box culvert satisfies the purpose and need of this project because of the structure's durability, ease and speed of construction, and economic advantages.

## **7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS**

### **7.1 Precast Concrete Box Culvert Design and Construction**

The proposed replacement structure will consist of a 74-foot-long precast concrete box culvert with slope tapered inlet and outlet walls. The box culvert will have 1-foot tall precast headwalls. To prevent undermining, the box culvert should have 2-foot tall inlet and outlet toe walls and riprap aprons. We anticipate the bottom slab of the box culvert will be embedded approximately 2 feet into the streambed. 2-foot thick riprap aprons should be constructed at the inlet and outlet and should be embedded a minimum of 6 inches into the streambed. The riprap aprons will be covered with the engineered streambed material to provide continuity of the natural streambed.

Due to the soft Glaciomarine Silts, it is recommended that the box culvert be constructed on a 2-foot thick layer of crushed stone reinforced with geogrid and wrapped in stabilization/reinforcement geotextile. The stabilization/reinforcement geotextile should be hand-deployed on the prepared soil subgrade prior to installing the geogrid-reinforced stone mat. The crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type Underdrain Backfill material. The crushed stone shall be placed in maximum 8-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor (method of compaction approximating 97 percent of AASHTO T-108 maximum dry density).

The geotextile shall meet Class 1 Stabilization/Reinforcement Geotextile meeting MaineDOT Standard Specification 722.01. Adjoining sections of the stabilization geotextile should be overlapped by a minimum of 1 foot.

Precast concrete box culverts are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The design shall be designed in accordance with MaineDOT Standard Specification 534 – Precast Structural Concrete, MaineDOT Bridge Design Guide (BDG) Section 8 – Buried Structures, and American Association of State Highway and Transportation Officials Load Resistance and Factor Design Bridge Design Specifications, 9<sup>th</sup> Edition, 2020.

The loading specified for the design of the box shall be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The precast concrete box culvert shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1 and LRFD Section 12. The design should use Soil Type 4 as presented in the MaineDOT BDG Section 3.6 to calculate earth loads and earth pressures from the soil envelope. The backfill properties are as follows:  $\phi = 32^\circ$ ,  $\gamma = 125$  pcf.

The excavation should be maintained so that the bedding layer and box culvert are constructed in-the-dry. The soil envelope and backfill shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. In no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density. The precast concrete box culvert shall be installed in conformance with MaineDOT BDG Section 8 and MaineDOT Standard Specification Section 534.

### **7.1.1 Precast Concrete Box Culvert Headwalls**

Concrete headwalls will be included in the culvert design to retain crushed stone slope protection and prevent stones from dropping or eroding into the waterway. Nominal 1-foot thick by 1-foot high concrete headwalls are recommended.

### **7.1.2 Precast Concrete Inlet and Outlet Walls**

The precast concrete box culvert's outlet and inlet walls will be slope-tapered at 2H:1V (maximum). The left and right outlet walls will share the same precast base slab. The sloped inlet and outlet walls are essentially retaining walls and shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The inlet and outlet walls shall be designed to resist lateral earth pressures, vehicular loads and forces resulting from creep, temperature and shrinkage deformations of the concrete box culvert. The inlet and outlet walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil ( $h_{eq}$ ) of 2.0 feet per LRFD Article 3.11.6.4. Passive pressure resulting from the embedment of the box culvert and walls with engineered streambed, or any other material shall not contribute to resisting forces.

Inlet and outlet walls that are fixed to the box culvert should be designed to resist movement using an at-rest earth pressure coefficient,  $K_o$ , of 0.47. Wingwalls sections that are independent of the box culvert and free to rotate should be designed using the Rankine active earth pressure coefficient,  $K_a$ , of 0.46 assuming a 2H:1V backslope. The active earth pressure coefficient will change if the backslope conditions are different.

### **7.1.3 Precast Concrete Inlet and Outlet Toe Walls**

Toe walls shall extend below the bottom slab connecting the left and right walls at the inlet and outlet of the box culvert to prevent undermining per MaineDOT BDG Section 8.3.1. The inlet and outlet toe walls should extend a minimum of 1 foot below the maximum depth of scour.

### **7.1.4 Bearing Resistance**

To provide a stable subgrade and mitigate consolidation settlement, it is recommended that the precast concrete box culvert be bedded on a 2-foot-thick layer of crushed stone that is reinforced with geogrid and wrapped in stabilization/reinforcement geotextile placed on the native soil subgrade with a bottom elevation of approximately El. 156.5.

For a precast concrete box culvert with a base width of 20 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 2 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 2 ksf. Due to the large size of the concrete box culvert base, controlling deflection and not bearing resistance may govern the design. The service limit state bearing resistance may govern the design. In no instance shall bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as  $0.3f'_c$ .

### **7.1.5 Modulus of Subgrade Reaction**

Large span precast box culverts can be viewed similarly to a mat foundation. A common approach to the design of precast box culverts is to use beam on elastic foundation theory to compute the soil-structure interaction and deflections.

The modulus of subgrade reaction relates the box culvert bearing pressure to settlement and is often used in soil-structure interaction analyses. The modulus of subgrade reaction is dependent on many factors including the material properties and thickness of the bearing soils, geometry of the box culvert, and the stiffness of the box culvert. The box culvert shall be designed using a modulus of subgrade reaction,  $k_s$ , equal to 30 pounds per cubic inch (pci).

## **7.2 Subgrade Preparation for Box Culvert**

The glaciomarine soils encountered in borings at the box subgrade elevation consisted of soft to medium stiff silts and clays. Any unsuitable soils (i.e. low strength silts and clays and loose sands), and all timber grillage that may be encountered at the subgrade elevation, should be excavated down to expose competent, firm material and replaced with compacted granular borrow. This recommendation should be included in the contract documents as a General Note.

The excavation will require care to maintain bottom stability and bearing capacity. The following items will be necessary to maintain a stable excavation and bearing surface:

- Construction phase dewatering is recommended to allow the bearing pad construction in-the-dry;
- Limit vibration-induced disturbance to the subgrade, to limit the risk of excavation bottom heave;
- Use of a smooth-edged bucket and careful grade control will be necessary to avoid over excavation and/or disturbance of the subgrade;
- The box culvert shall be installed on a 2-foot thick layer of crushed stone be wrapped in stabilization/reinforcement geotextile;
- Hand-deploy the geotextile on the prepared soil subgrade prior to installing the geogrid-reinforced stone mat;
- Steel rollers and steel plates can be utilized to move precast units on the geotextile. Alternatively, the crushed stone thickness of the geogrid-reinforced mat can be reduced to 18" and the wrapped stone mat topped with 6-inches of granular borrow to facilitate setting and sliding precast box segments.

The crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type C Underdrain Backfill material. The crushed stone shall be placed in maximum 8-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind plate compactor.

### **7.3 Settlement**

The proposed box culvert will bear on Glaciomarine clays and silts, underlain by Marine Sands. These soils will undergo immediate and consolidation settlement in response to a net increase of vertical overburden pressure. Based on an estimated service limit state pressure of 1,250 psf for a 20-foot wide precast concrete box, an immediate settlement on the order of 1.5-inch is estimated. An additional 2-inches of long-term consolidation is anticipated over the next 50-years.

### **7.4 Frost Protection**

Foundations placed on the fill or native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, West Gardiner has a design freezing index (DFI) of approximately 1600 F-degree days. A water content of 15% was used for fine-grained soils. These components correlate to a frost depth of 6.5 feet.

It is recommended that foundations bearing on soil be designed with an embedment of approximately 6.5 feet for frost protection.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

### **7.5 Scour and Riprap**

The box culvert shall be constructed with integral concrete headwalls and inlet and outlet walls to retain stone slopes and prevent stone slope protection from dropping or eroding into the waterway. Inlet and outlet toe walls shall be provided that extend a minimum of 1-foot below the maximum depth of scour. Inlet and outlet toe walls shall also be protected with riprap aprons.

Where required, slopes shall be armored with a 3-foot thick layer of riprap conforming to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1-foot below the streambed elevation. The riprap slopes shall be constructed no steeper than 1.75H:1V extending from the edge of the roadway down to the existing ground surface.

## **7.6 Seismic Design Considerations**

In conformance with LRFD Article 3.10.1, seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine; therefore, seismic analysis is not required.

## **8.0 CONSTRUCTION CONSIDERATIONS**

The soil envelope and backfill for the box culvert shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The granular borrow backfill should be placed in lifts of 6- to 8-inches-thick loose measure and compacted to the manufacturer's specifications. To minimize future settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

The box culvert will be constructed on a 2-foot thick layer of crushed stone reinforced with geogrid and wrapped in stabilization/reinforcement geotextile. The geotextile should be hand-deployed on the prepared soil subgrade prior to installing the geogrid-reinforced stone mat. The crushed stone shall meet the requirements of MaineDOT Standard Specification 703.22 – Type Underdrain Backfill material. The crushed stone shall be placed in maximum 8-inch thick lifts and each lift compacted with at least 4 passes of a walk-behind vibrator-type compactor. The geotextile shall meet Class 1 Stabilization/Reinforcement Geotextile meeting MaineDOT Standard Specification 722.01. Adjoining sections of the stabilization geotextile should be overlapped by a minimum of 1-foot.

The following items will be necessary to maintain a stable excavation and bearing surface:

- Construction phase dewatering is recommended to allow the bearing pad construction in the dry. Cofferdams may be required to divert flow away from the new culvert location during construction;
- The contractor shall not operate heavy equipment over the excavated subgrade to minimize subgrade disturbance;
- Limit vibration-induced disturbance to limit the risk of excavation bottom heave;
- Use of a smooth-edged bucket and careful grade control will be necessary to avoid over excavation and/or disturbance of the subgrade;
- Hand-deploy the stabilization/reinforcement geotextile on the prepared soil subgrade prior to installing the crushed stone mat;
- Steel rollers and steel plates can be utilized to move precast units on the geotextile. Alternatively, the crushed stone thickness of the geogrid-reinforced mat can be reduced to 18" and the wrapped stone mat topped with 6-inches of granular borrow to facilitate setting and sliding precast box segments.

The Contractor shall minimize disturbance to the silt and clay subgrade surface and protect the subgrade surface from any unnecessary construction traffic. Any soft soils or organic material encountered at the bearing elevation shall be removed and replaced with compacted Granular Borrow – Material for Underwater Backfill.

Earthwork and excavations will result in the exposure of clays, silts or other organics. These soils may be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace the disturbed materials with compacted Granular Borrow – Material for Underwater Backfill.

Soils may become saturated and water seepage may be encountered during construction and in excavations. There may be localized sloughing and instability in some excavations and cut slopes. The Contractor should control groundwater and surface water infiltration using temporary ditches, sump pumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert groundwater and surface water.

## **9.0 CLOSURE**

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Gosline Bridge in West Gardiner, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

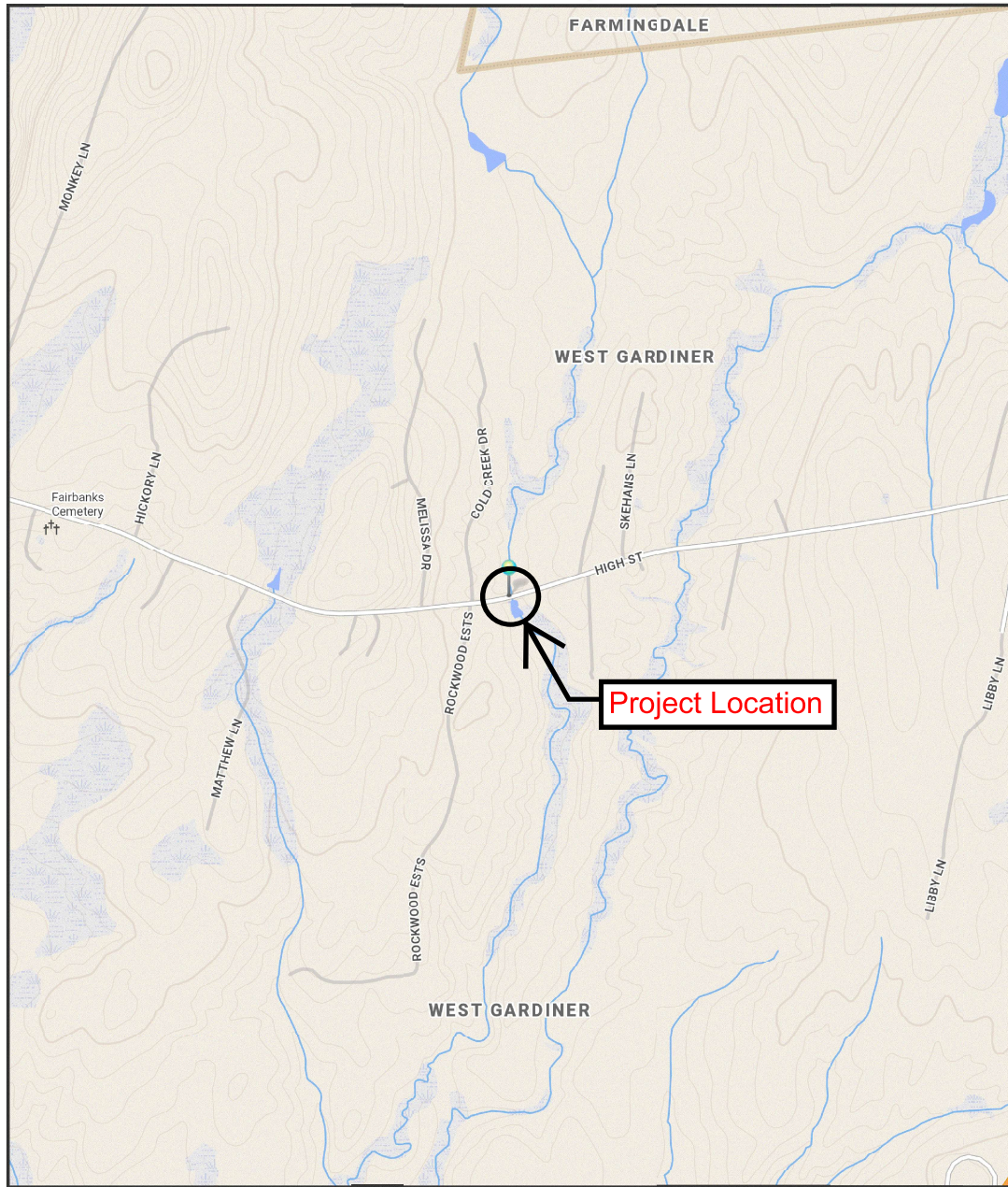
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. These analyses and recommendations are based in part upon limited subsurface investigations at discrete exploratory locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

It is recommended that the geotechnical engineer be provided the opportunity for a review of the design and specifications so that the earthwork and foundation recommendations and construction considerations in the report are properly interpreted and implemented in the design and specifications.

## **Sheets**



## WEST GARDINER, MAINE



The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch.

0.25 Miles  
1 inch = 0.28 miles

Date: 9/14/2021  
Time: 9:41:53 AM

<p>SHEET NUMBER</p> <p><b>1</b></p> <p>OF 3</p>	<p>GOSLINE BRIDGE COLD STREAM</p> <p>WEST GARDINER      KENNEBEC COUNTY</p> <p><b>LOCATION MAP</b></p>	<p>STATE OF MAINE DEPARTMENT OF TRANSPORTATION</p> <p><b>2309000</b></p> <p><b>WIN</b></p> <p>BRIDGE NO. 2321      <b>23090.00</b>      BRIDGE PLANS</p>
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## **Appendix A**

### Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM				
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS  (more than half of material is larger than No. 200 sieve size)	GRAVELS  (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS  (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS  (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS  (little or no fines)	SW	Well-graded sands, Gravelly sands, little or no fines
			SP	Poorly-graded sands, Gravelly sand, little or no fines.
SANDS WITH FINES (Appreciable amount of fines)		SM	Silty sands, sand-silt mixtures	
	SC	Clayey sands, sand-clay mixtures.		
FINE-GRAINED SOILS  (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS  (liquid limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.	
		CL	Inorganic clays of low to medium plasticity, Gravelly clays, Sandy clays, Silty clays, lean clays.	
		OL	Organic silts and organic Silty clays of low plasticity.	
	SILTS AND CLAYS  (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.	
		CH	Inorganic clays of high plasticity, fat clays.	
		OH	Organic clays of medium to high plasticity, organic silts.	
	HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.	
<b>Desired Soil Observations (in this order, if applicable):</b> Color (Munsell color chart) Moisture (dry, damp, moist, wet) Density/Consistency (from above right hand side) Texture (fine, medium, coarse, etc.) Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc., ) Cementation (weak, moderate, or strong) Geologic Origin (till, marine clay, alluvium, etc.) Groundwater level				
<b>Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information</b>				

MODIFIED BURMISTER SYSTEM			
<u>Descriptive Term</u> trace little some adjective (e.g. Sandy, Clayey)		<u>Portion of Total (%)</u> 0 - 10 11 - 20 21 - 35 36 - 50	
<b>TERMS DESCRIBING DENSITY/CONSISTENCY</b>			
<b>Coarse-grained soils</b> (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value).			
<u>Density of Cohesionless Soils</u> Very loose Loose Medium Dense Dense Very Dense		<u>Standard Penetration Resistance N-Value (blows per foot)</u> 0 - 4 5 - 10 11 - 30 31 - 50 > 50	
<b>Fine-grained soils</b> (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) Gravelly, Sandy or Silty clays; and (3) Clayey silts. Consistency is rated according to undrained shear strength as indicated.			
<u>Consistency of Cohesive soils</u> Very Soft Soft Medium Stiff  Stiff  Very Stiff Hard	<u>SPT N-Value (blows per foot)</u> WOH, WOR, WOP, <2 2 - 4 5 - 8  9 - 15  16 - 30 >30	<u>Approximate Undrained Shear Strength (psf)</u> 0 - 250 250 - 500 500 - 1000  1000 - 2000  2000 - 4000 over 4000	<u>Field Guidelines</u> Fist easily penetrates Thumb easily penetrates Thumb penetrates with moderate effort Indented by thumb with great effort Indented by thumbnail Indented by thumbnail with difficulty
<b>Rock Quality Designation (RQD):</b> RQD (%) = <u>sum of the lengths of intact pieces of core* &gt; 4 inches</u> length of core advance *Minimum NQ rock core (1.88 in. OD of core)			
<b>Rock Quality Based on RQD</b> <u>Rock Quality</u> <u>RQD (%)</u> Very Poor      ≤25 Poor      26 - 50 Fair      51 - 75 Good      76 - 90 Excellent      91 - 100			
<b>Desired Rock Observations (in this order, if applicable):</b> Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -tightness (tight, open, or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock quality (very poor, poor, etc.) ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12 Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))			
<b>Sample Container Labeling Requirements:</b> WIN      Blow Counts Bridge Name / Town      Sample Recovery Boring Number      Date Sample Number      Personnel Initials Sample Depth			

<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS				<b>Project:</b> Gosline Bridge #2321 carries High Street over Cold Stream <b>Location:</b> West Gardiner, Maine				<b>Boring No.:</b> BB-WGCS-101 <b>WIN:</b> 23090.00																																																																																																																																																																																																																																																																									
<b>Driller:</b> MaineDOT				<b>Elevation (ft.)</b> 173.4				<b>Auger ID/OD:</b> 5" Solid Stem																																																																																																																																																																																																																																																																									
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<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS				<b>Project:</b> Gosline Bridge #2321 carries High Street over Cold Stream <b>Location:</b> West Gardiner, Maine				<b>Boring No.:</b> BB-WGCS-101  <b>WIN:</b> 23090.00																																							
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Page 2 of 2  
  
**Boring No.:** BB-WGCS-101

<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS				<b>Project:</b> Gosline Bridge #2321 carries High Street over Cold Stream <b>Location:</b> West Gardiner, Maine				<b>Boring No.:</b> BB-WGCS-102 <b>WIN:</b> 23090.00									
<b>Driller:</b> MaineDOT				<b>Elevation (ft.)</b> 171.7				<b>Auger ID/OD:</b> 5" Solid Stem									
<b>Operator:</b> Daggett/Aaron/Niles				<b>Datum:</b> NAVD88				<b>Sampler:</b> Standard Split Spoon									
<b>Logged By:</b> B. Wilder				<b>Rig Type:</b> CME 45C				<b>Hammer Wt./Fall:</b> 140/3/30"									
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<b>Boring Location:</b> 3+74.2, 7.6 ft Lt.				<b>Casing ID/OD:</b> NW-3"				<b>Water Level*:</b> 2.5 ft bgs.									
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Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				Su = Peak/Remolded Field Vane Undrained Shear Strength (psf) Su(lab) = Lab Vane Undrained Shear Strength (psf) qp = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N60 = SPT N-uncorrected Corrected for Hammer Efficiency N60 = (Hammer Efficiency Factor/60%)*N-uncorrected									
Tv = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																	
<b>Sample Information</b>												<b>Graphic Log</b>		<b>Visual Description and Remarks</b>		<b>Laboratory Testing Results/ AASHTO and Unified Class.</b>	
<b>Depth (ft.)</b>	<b>Sample No.</b>	<b>Pen./Rec. (in.)</b>	<b>Sample Depth (ft.)</b>	<b>Blows (6 in.) Shear Strength (psf) or RQD (%)</b>	<b>N-uncorrected</b>	<b>N60</b>	<b>Casing Blows</b>	<b>Elevation (ft.)</b>									
0								SSA	171.1		7" HMA.						
5	1D	24/14	5.00 - 7.00	6/8/7/5	15	22					Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).						
10	2D	24/18	10.00 - 12.00	4/4/4/4	8	12	17		161.2		Dark brown, moist, medium dense, Silty fine to medium SAND, some organics, some timber cribbing, (Fill)						
15																	
	3D	24/24	16.00 - 18.00	WOH/WOH/WOH/ WOH	---		21		156.2		Casing sunk to 15.5 ft bgs while washing out. Grey, wet, soft to medium stiff, SILT, some clay, trace gravel, trace fine sand, (Glaciomarine). 55x110 mm vane raw torque readings: V1: 12.0/2.0 ft-lbs V2: 11.0/4.0 ft-lbs			G#337374 A-6, CL WC=40.6% LL=35 PL=21 PI=14			
	V1		16.63 - 17.00	Su=536/89 psf													
	V2		17.63 - 18.00	Su=491/179 psf			23										
	4D	24/20	18.00 - 20.00	WOH/WOH/WOH/ WOH	---		13										
	V3		18.63 - 19.00	Su=536/134 psf													
	V4		19.63 - 20.00	Su=536/89 psf			13										
20	5D	19.2/13	20.00 - 21.60	WOH/2/11/40	13	19	14		151.2		Grey, wet, medium stiff, Silty CLAY, trace sand, (Glaciomarine). V3: 12.0-3.0 ft-lbs V4: 12.0/2.0 ft-lbs						
	R1	60/56	21.60 - 26.60	RQD = 85%			a50 NQ-2		150.1		Brown, wet, medium dense, Gravelly fine to coarse SAND, some silt (Marine Sand). a50 blows for 0.6 ft.						
25											Top of Bedrock at Elev. 150.1 ft. R1: Bedrock: Purplish to greenish white, banded, fine grained GRANOFELS (GNEISS), very hard, fresh, weakly foliated at low to moderate angles to massive. [Vassalboro Formation]						
<b>Remarks:</b>																	
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.												Page 1 of 2					
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.												Boring No.: BB-WGCS-102					

<div>Maine Department of Transportation</div> <div>Soil/Rock Exploration Log</div> <div>US CUSTOMARY UNITS</div>						<div>Project:</div> Gosline Bridge #2321 carries High Street over Cold Stream <div>Location:</div> West Gardiner, Maine		<div>Boring No.:</div> BB-WGCS-102																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
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<div>Driller:</div> MaineDOT				<div>Elevation (ft.)</div> 171.7		<div>Auger ID/OD:</div> 5" Solid Stem																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
<div>Operator:</div> Daggett/Aaron/Niles				<div>Datum:</div> NAVD88		<div>Sampler:</div> Standard Split Spoon																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
<div>Logged By:</div> B. Wilder				<div>Rig Type:</div> CME 45C		<div>Hammer Wt./Fall:</div> 140/3/30"																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
<div>Date Start/Finish:</div> 10/30/2019; 08:00-13:00				<div>Drilling Method:</div> Cased Wash Boring		<div>Core Barrel:</div> NQ-2"																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
<div>Boring Location:</div> 3+74.2, 7.6 ft Lt.				<div>Casing ID/OD:</div> NW-3"		<div>Water Level*:</div> 2.5 ft bgs.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
<div>Hammer Efficiency Factor:</div> 0.886				<div>Hammer Type:</div> <div>Automatic<input checked="" type="checkbox"/></div> <div>Hydraulic<input type="checkbox"/></div> <div>Rope &amp; Cathead<input type="checkbox"/></div>																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
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[Vassalboro Formation] Rock Quality = Excellent R2: Core Times (min:sec) No Core Times Taken. 100% Recovery</td><td rowspan="12"></td></tr><tr><td></td><td>R2</td><td>60/60</td><td>26.60 - 31.60</td><td>RQD = 95%</td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>30</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="12"></td><td rowspan="12">Bottom of Exploration at 31.6 feet below ground surface.</td><td 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## **Appendix B**

### Rock Core Photographs



**Gosline Bridge #2321 Carries High Street Over Cold Brook**  
**West Gardiner, ME**  
*Rock Core Photographs*

Boring No.	Run	Depth (ft)	Pentration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-WGCS-101	R1	22.0-27.0	60	59	55	92	GRANOFELS (GNEISS)	1
BB-WGCS-101	R2	27.0-32.0	60	60	58	97	GRANOFELS (GNEISS)	2
BB-WGCS-102	R1	21.6-26.6	60	56	51	85	GRANOFELS (GNEISS)	3
BB-WGCS-102	R2	26.6-31.6	60	60	57	95	GRANOFELS (GNEISS)	4



**Notes:** 1. "Box row" indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

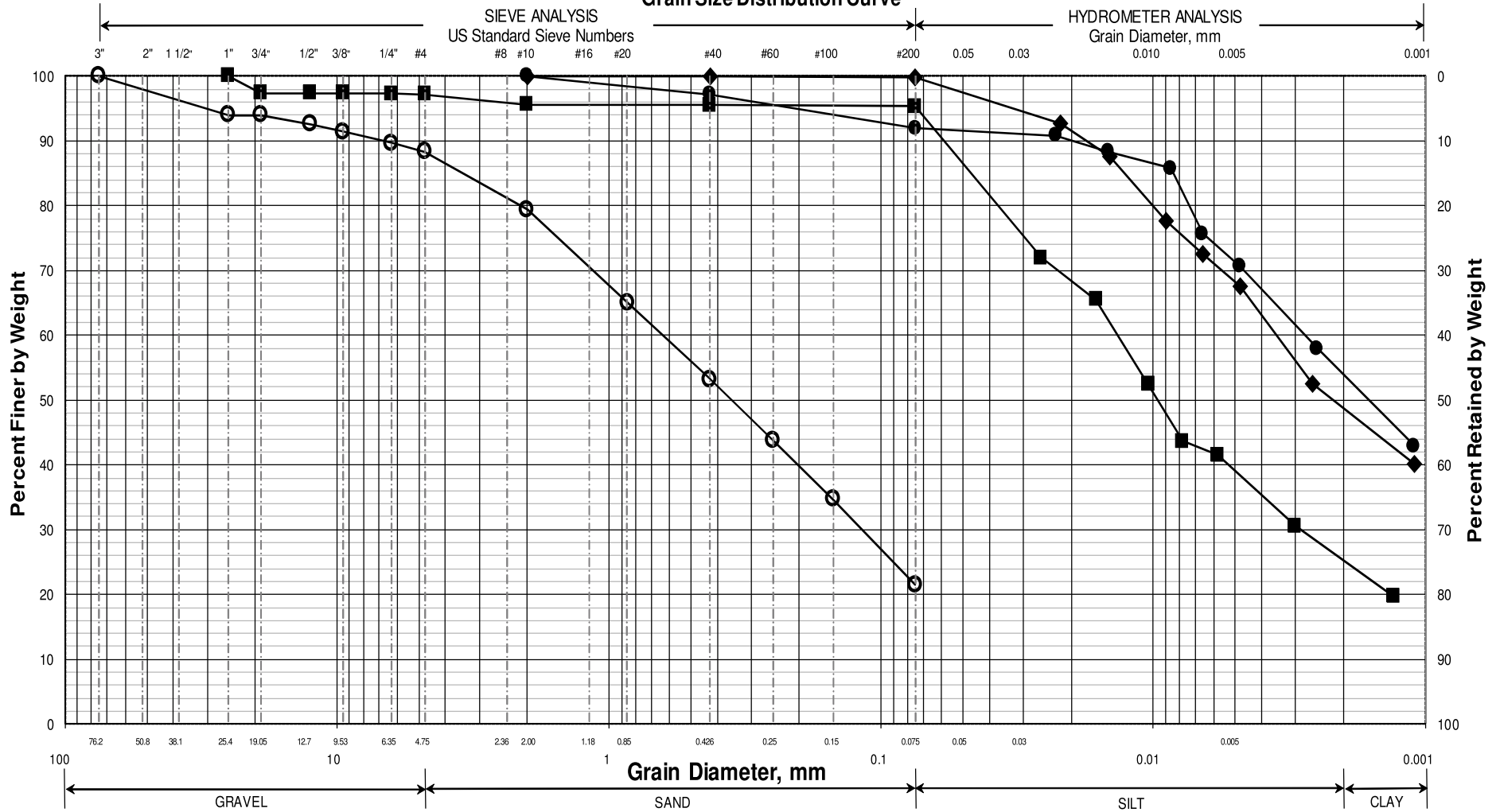
## **Appendix C**

### Laboratory Test Results

**Work Number: 23090.00**

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

# Maine Department of Transportation Grain Size Distribution Curve



## UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-WGCS-101/1D	3+46.4	8.3 RT	5.0-7.0	SAND, some silt, little gravel.	12.4			
◆	BB-WGCS-101/2D	3+46.4	8.3 RT	11.3-14.0	Clayey SILT, trace sand.	35.1	38	22	16
■	BB-WGCS-102/3D	3+74.2	7.6 LT	16.0-18.0	SILT, some clay, trace gravel, trace sand.	40.6	35	21	14
●	BB-WGCS-102/4D	3+74.2	7.6 LT	18.0-20.0	Silty CLAY, trace sand.	42.7	29	22	7
▲									
×									

WIN	
023090.00	
Town	
West Gardiner	
Reported by/Date	
WHITE, TERRY A	3/29/2021



# GEOTECHNICAL TEST REPORT

## Central Laboratory

## SAMPLE INFORMATION

Reference No.

Boring No./Sample No.

## Sample Description

Sampled

Received

**337372**

# BB-WGCS-101/2D

### GEOTECHNICAL (DISTURBED)

**10/30/2019**

11/4/2019

Sample Type: **GEOTECHNICAL**      Location:

Station: **3+46.4**      Offset, ft: **8.3**      RT Dbfg, ft: **11.3-14.0**

**WIN/Town 023090.00 - WEST GARDINER**

Sampler: **BRUCE WILDER**

## TEST RESULTS

### Sieve Analysis (T 88)

### Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	<b>100.0</b>
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	<b>99.9</b>
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	<b>99.8</b>
[0.0220 mm]	<b>92.7</b>
[0.0144 mm]	<b>87.6</b>
[0.0090 mm]	<b>77.6</b>
[0.0066 mm]	<b>72.6</b>
[0.0048 mm]	<b>67.6</b>
[0.0026 mm]	<b>52.6</b>
[0.0011 mm]	<b>40.1</b>

## Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	38
Plastic Limit (T 90), %	22
Plasticity Index (T 90), %	16
Specific Gravity, Corrected to 20°C (T 100)	2.78
Loss on Ignition, % (T 267)	
Water Content (T 265), %	35.1

## Consolidation (T 216)

Trimmings, Water Content, %			
	Initial	Final	
Water Content, %			Pmin
Dry Density, lbs/ft³			Pp
Void Ratio			Pmax
Saturation, %			Cc/C'c

## Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

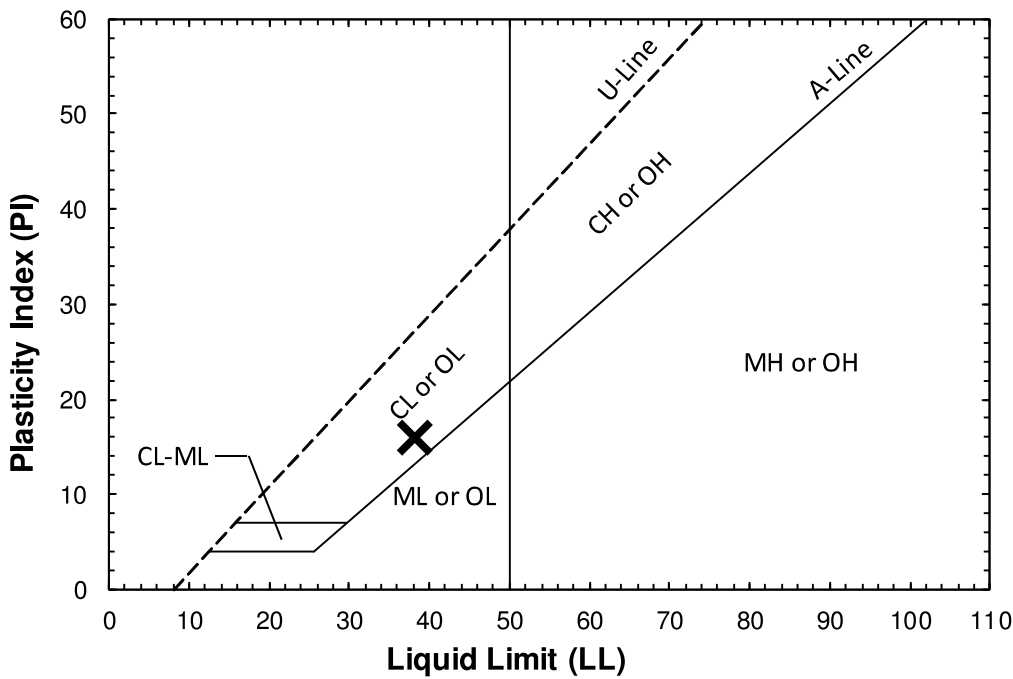
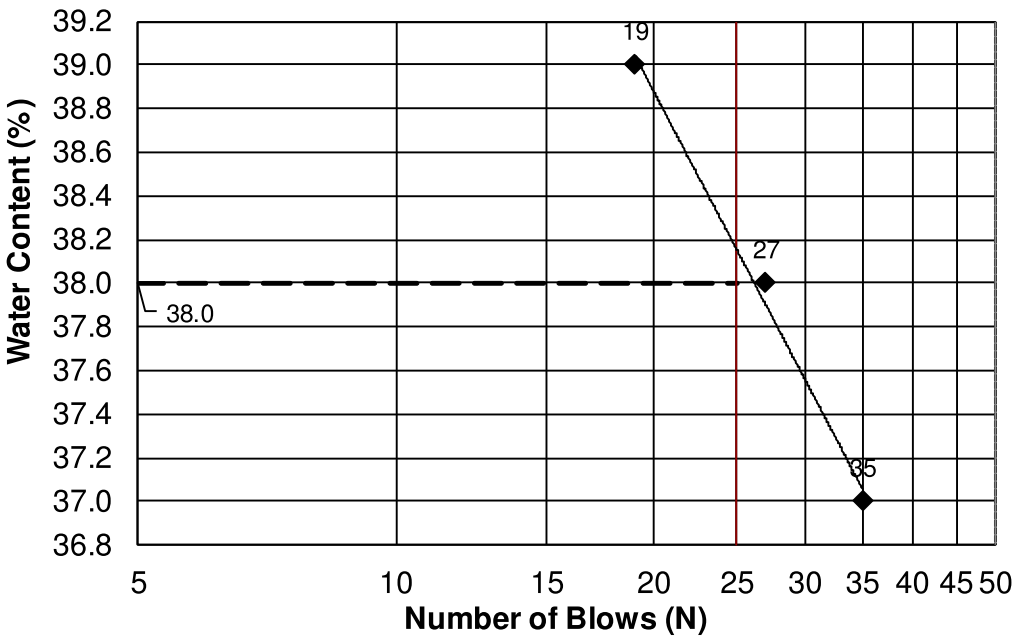
## AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: 11/12/2019

*Paper Copy: Lab File: Project File: Geotech File*

TOWN	West Gardiner	Reference No.	337372
WIN	023090.00	Water Content, %	35.1
Sampled	10/30/2019	Liquid Limit @ 25 blows (T 89), %	38
Boring No./Sample No.	BB-WGCS-101/2D	Plastic Limit (T 90), %	22
Station	3+46.4	Plasticity Index (T 90), %	16
Depth	11.3-14.0	Tested By	BBURR





# GEOTECHNICAL TEST REPORT

## Central Laboratory

### SAMPLE INFORMATION

Reference No. **337374** Boring No./Sample No. **BB-WGCS-102/3D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **10/30/2019** Received **11/4/2019**

Sample Type: **GEOTECHNICAL** Location: Station: **3+74.2** Offset, ft: **7.6** LT Dbfg, ft: **16.0-18.0**

WIN/Town **023090.00 - WEST GARDINER** Sampler: **BRUCE WILDER**

### TEST RESULTS

#### Sieve Analysis (T 88)

##### Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	<b>100.0</b>
¾ in. [19.0 mm]	<b>97.4</b>
½ in. [12.5 mm]	<b>97.4</b>
⅜ in. [9.5 mm]	<b>97.4</b>
¼ in. [6.3 mm]	<b>97.3</b>
No. 4 [4.75 mm]	<b>97.2</b>
No. 10 [2.00 mm]	<b>95.6</b>
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	<b>95.5</b>
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	<b>95.3</b>
[0.0258 mm]	<b>72.0</b>
[0.0162 mm]	<b>65.5</b>
[0.0104 mm]	<b>52.4</b>
[0.0078 mm]	<b>43.6</b>
[0.0058 mm]	<b>41.5</b>
[0.0030 mm]	<b>30.5</b>
[0.0013 mm]	<b>19.6</b>

#### Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	<b>35</b>
Plastic Limit (T 90), %	<b>21</b>
Plasticity Index (T 90), %	<b>14</b>
Specific Gravity, Corrected to 20°C (T 100)	<b>2.72</b>
Loss on Ignition, % (T 267)	
Water Content (T 265), %	<b>40.6</b>

#### Consolidation (T 216)

Trimmings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

#### Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

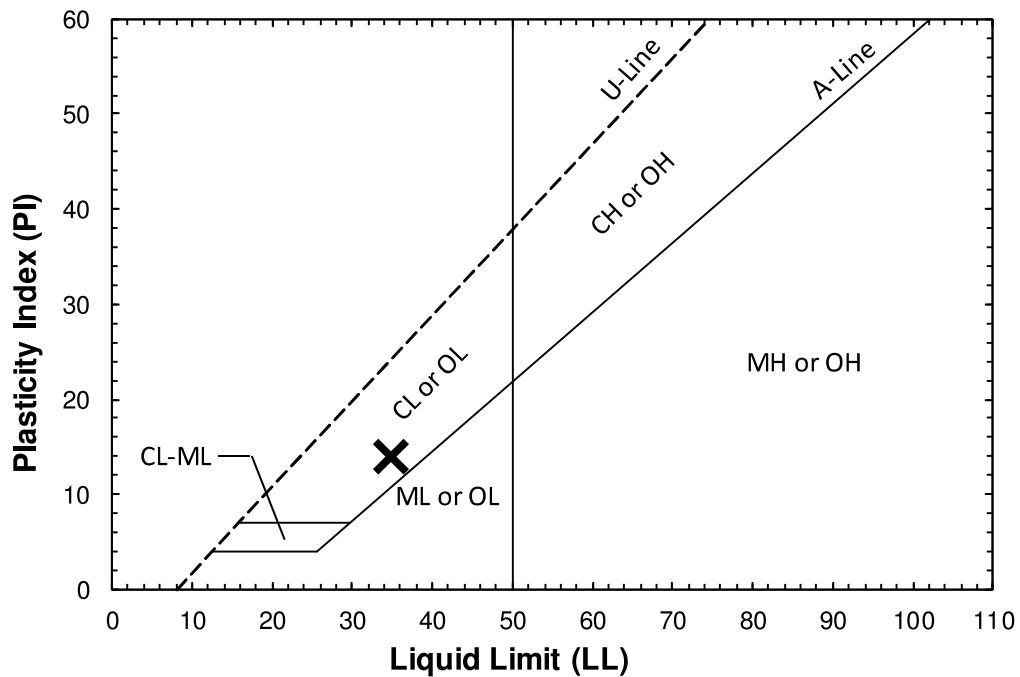
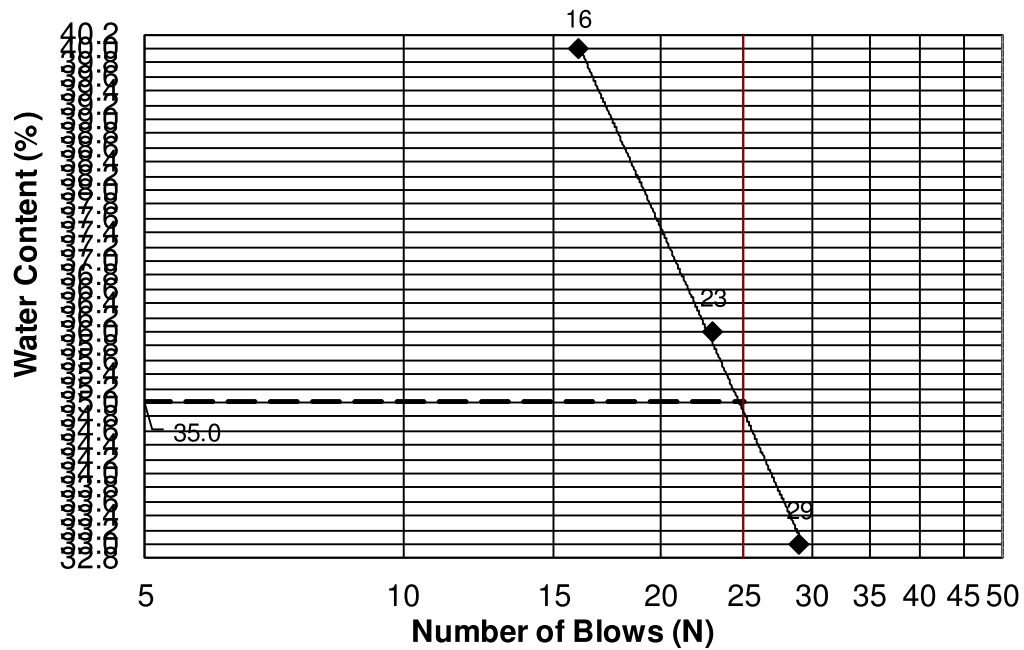
### AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **11/12/2019**

Paper Copy: Lab File; Project File; Geotech File

TOWN	West Gardiner	Reference No.	337374
WIN	023090.00	Water Content, %	40.6
Sampled	10/30/2019	Liquid Limit @ 25 blows (T 89), %	35
Boring No./Sample No.	BB-WGCS-102/3D	Plastic Limit (T 90), %	21
Station	3+74.2	Plasticity Index (T 90), %	14
Depth	16.0-18.0	Tested By	BBURR





# GEOTECHNICAL TEST REPORT

## Central Laboratory

### SAMPLE INFORMATION

Reference No. **337375** Boring No./Sample No. **BB-WGCS-102/4D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **10/30/2019** Received **11/4/2019**

Sample Type: **GEOTECHNICAL** Location: Station: **3+74.2** Offset, ft: **7.6** LT Dbfg, ft: **18.0-20.0**

WIN/Town **023090.00 - WEST GARDINER** Sampler: **BRUCE WILDER**

### TEST RESULTS

#### Sieve Analysis (T 88)

##### Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	<b>100.0</b>
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	<b>97.2</b>
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	<b>92.0</b>
[0.0227 mm]	<b>90.8</b>
[0.0146 mm]	<b>88.3</b>
[0.0086 mm]	<b>85.8</b>
[0.0066 mm]	<b>75.7</b>
[0.0048 mm]	<b>70.6</b>
[0.0025 mm]	<b>58.0</b>
[0.0011 mm]	<b>42.9</b>

#### Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	<b>29</b>
Plastic Limit (T 90), %	<b>22</b>
Plasticity Index (T 90), %	<b>7</b>
Specific Gravity, Corrected to 20°C (T 100)	<b>2.77</b>
Loss on Ignition, % (T 267)	
Water Content (T 265), %	<b>42.7</b>

#### Consolidation (T 216)

Trimming, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

#### Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

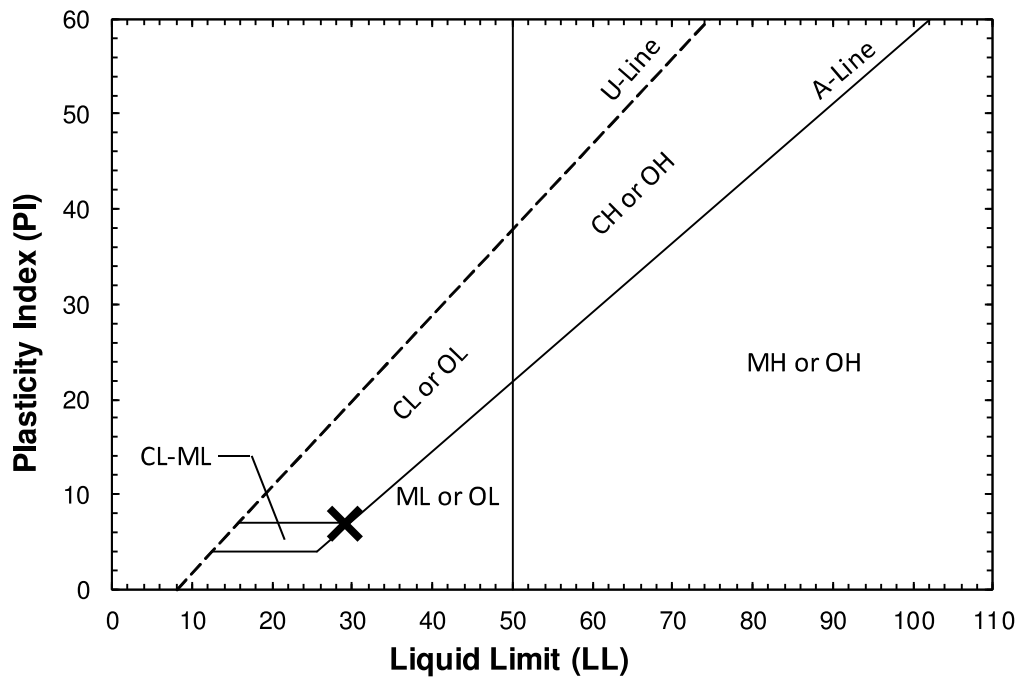
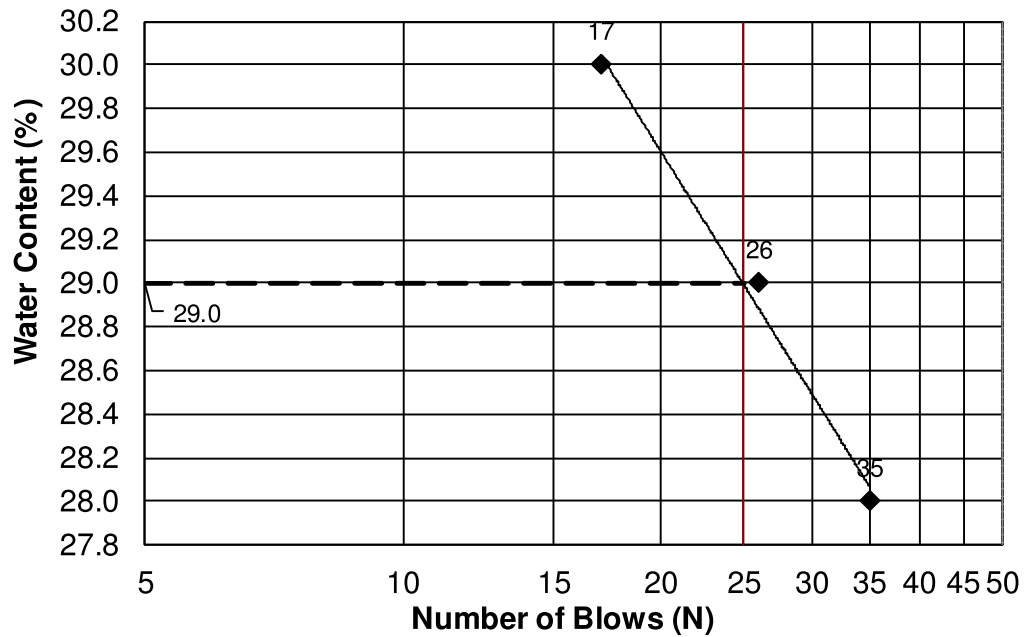
### AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **11/12/2019**

Paper Copy: Lab File; Project File; Geotech File

TOWN	West Gardiner	Reference No.	337375
WIN	023090.00	Water Content, %	42.7
Sampled	10/30/2019	Liquid Limit @ 25 blows (T 89), %	29
Boring No./Sample No.	BB-WGCS-102/4D	Plastic Limit (T 90), %	22
Station	3+74.2	Plasticity Index (T 90), %	7
Depth	18.0-20.0	Tested By	BBURR



## **Appendix D**

### Calculations

## Liquidity Index and Sensitivity

**Liquidity Index**

$$LI := \frac{WC - PL}{LL - PL}$$

Das, Principles of Engineering, 7th Edition,  
Equation 4.16

**BB-WGSC-101, 2D**

$$WC := 35.1$$

$$LL := 38$$

$$PL := 22$$

$$LI := \frac{WC - PL}{LL - PL} = 0.82$$

**BB-WGSC-102, 3D**

$$WC := 40.6$$

$$LL := 35$$

$$PL := 21$$

$$LI := \frac{WC - PL}{LL - PL} = 1.4$$

**BB-WGSC-102, 4D**

$$WC := 42.7$$

$$LL := 29$$

$$PL := 22$$

$$LI := \frac{WC - PL}{LL - PL} = 2.96$$

**Sensitivity**

**BB-WGCS-101/V1**

$$Su := 1205\text{psf}$$

$$Su_{re} := 268\text{psf}$$

$$\frac{Su}{Su_{re}} = 4.5$$

**BB-WGCS-101/V2**

$$Su := 1161\text{psf}$$

$$Su_{re} := 223\text{psf}$$

$$\frac{Su}{Su_{re}} = 5.21$$

**BB-WGCS-102/V1**

$$Su := 536\text{psf}$$

$$Su_{re} := 89\text{psf}$$

$$\frac{Su}{Su_{re}} = 6.02$$

**BB-WGCS-102/V2**

$$Su := 491\text{psf}$$

$$Su_{re} := 179\text{psf}$$

$$\frac{Su}{Su_{re}} = 2.74$$

**BB-WGCS-102/V3**

$$Su := 536\text{psf}$$

$$Su_{re} := 134\text{psf}$$

$$\frac{Su}{Su_{re}} = 4$$

**BB-WGCS-102/V4**

$$Su := 536\text{psf}$$

$$Su_{re} := 89\text{psf}$$

$$\frac{Su}{Su_{re}} = 6.02$$

Sensitivities range from 2.7 to 6.0, ranging from moderately sensitive to sensitive

Fang, Foundation Engineering  
Handbook 3.13.3

## Earth Pressure

## Earth Pressure:

### Backfill engineering strength parameters

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

Unit weight  $\gamma := 125 \cdot \text{pcf}$

Internal friction angle  $\phi := 32 \cdot \text{deg}$

Cohesion  $c := 0 \cdot \text{psf}$

### Outlet Walls Fixed to Box

#### At-Rest Earth Pressure - Rankine Theory

$$K_o := 1 - \sin(\phi)$$

$$K_o = 0.47$$

Fang, Foundation  
Engineering Handbook  
2nd ed. Pg. 224, Eq. 6.2  
Formula for normally  
consolidated soils.

### Outlet walls free to rotate - Active Earth Pressure - Rankine Theory

The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

For cantilver walls with horizontal backslope:

$$K_{ar} := \tan\left(45 \cdot \text{deg} - \frac{\phi}{2}\right)^2$$

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

$\beta$  = Angle of fill slope to the horizontal

$$\beta := 26.56 \cdot \text{deg}$$

$$K_{ar\_slope} := \cos(\beta) \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

$$K_{ar\_slope} = 0.46$$

$P_a$  is oriented at an angle of  $\beta$  to the vertical plane - See MaineDOT Bridge Design Guide Figure 3-3 attached.

## 6.1 AT-REST LATERAL PRESSURES

At-rest pressures exist in level ground, and develop under long-term conditions as the soil is deposited and acted upon by changes in the loading environment as caused by erosion, glaciers, and physicochemical processes. At-rest pressures rigorously only apply for walls that are placed into the ground with a minimum of disturbance and that remain unmoved during loading, or for unmoving, frictionless walls with a backfill placed with a minimum of compactive effort. In practice such conditions are rarely achieved. However, at-rest pressures are still useful in design as either a baseline against which other pressure states can be judged or as an assumed conservative choice for the design loading.

At-rest effective lateral pressures are often assumed to follow a linear distribution (Fig. 6.2), with the effective lateral pressure  $\sigma'_x$  taken as a simple multiple of the vertical effective pressure  $\sigma'_z$ :

$$\sigma'_x = K_0(\sigma'_z) \quad (6.1)$$

In homogeneous, dry soil with a constant  $K_0$  and unit weight, both the vertical and lateral pressures are linearly distributed. With the presence of a water table, the at-rest pressure distribution exhibits a break in slope at the water table, reflecting the use of submerged unit weights to determine vertical effective stresses (Fig. 6.2).

Our early concepts of the parameter  $K_0$  were formed on the basis of normally consolidated soils. Jaky (1944) proposed a relationship between  $K_0$  and the drained friction angle  $\phi'$  for normally consolidated soils:

$$K_0 = 1 - \sin \phi' \quad (6.2)$$

Numerous studies have confirmed the general validity of this empirical equation (Brooker and Ireland, 1965; Mayne and Kulhawy, 1982). However, results from laboratory experiments and in-situ tests have shown that the  $K_0$  value also varies as a function of overconsolidation ratio (OCR) and stress history. For the case of a soil that has been subjected to one or more cycles of unloading, Schmidt (1966) proposed that  $K_0$  can be determined as a function of its value in the normally consolidated state using the relationship

$$K_{0u} = K_{0nc}(\text{OCR})^\alpha \quad (6.3)$$

in which  $K_{0u}$  is the coefficient for unloading,  $K_{0nc}$  is the coefficient for the normally consolidated soil, and  $\alpha$  is a dimensionless coefficient. Experimental data have confirmed this relationship, and Mayne and Kulhawy (1982) showed that, for most soils,  $\alpha$  can be taken as  $\sin \phi'$ .

Soils that are overconsolidated and are in the process of being reloaded pose a difficulty in that Equation 6.3 does not apply. For this condition, a more complex equation is needed as well as a full knowledge of the stress history of the soil (Mayne and Kulhawy, 1982). For practical purposes, it may

**TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.**

Soil type	Coefficient of Lateral Earth Pressure			
	OCR = 1	OCR = 2 <sup>a</sup>	OCR = 5 <sup>a</sup>	OCR = 10 <sup>a</sup>
Loose sand	0.45	0.65	1.10	1.50
Medium sand	0.40	0.60	1.05	1.55
Dense sand	0.35	0.55	1.00	1.50
Silt	0.50	0.70	1.10	1.60
Lean clay, CL	0.60	0.80	1.20	1.65
Highly plastic clay, CH	0.65	0.80	1.10	1.40

<sup>a</sup> Unloading cycle.

be enough to know that the  $K_0$  during reloading falls about halfway between that for unloading and normally consolidated conditions. Also,  $K_0$  might be directly determined through in-situ testing methods.

Table 6.1 presents typical values for  $K_0$  for a subset of soils. For other conditions,  $K_0$  values can be determined directly from Equations 6.2 and 6.3, and/or using in-situ testing techniques.

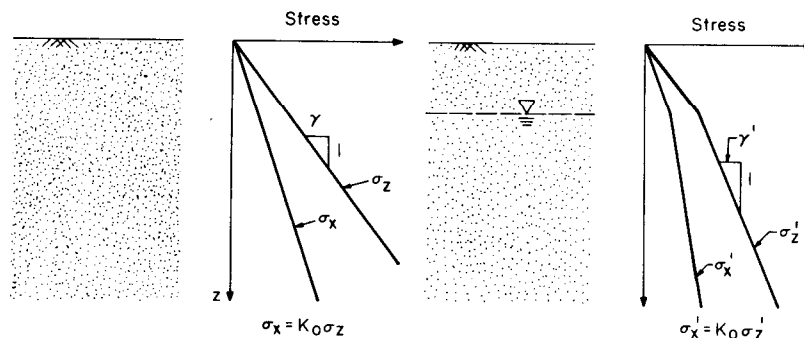
Because the  $K_0$  value in a given soil often varies with depth, and the soil types themselves may change with depth, the at-rest lateral pressure distribution is typically not linear as shown in Figure 6.2. Self-boring pressuremeter tests in clays with overconsolidated profiles induced by desiccation have demonstrated that the  $K_0$  under such conditions decreases with depth in the soil deposit and reaches a steady state where the desiccation effects are no longer present (Clough and Denby, 1980).

## 6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

Most walls move, either by global shifting or by local deformations. These movements cause adjustments to occur in the earth loads and the pressure distributions. Conventional means for assessing the effects of system movements are to set them into the context of extreme conditions. These are referred to as the active and passive earth pressure loadings.

### 6.2.1 Active Pressure

Assuming that a gravity wall with no friction on its face is translated away from a soil mass that is initially at the at-rest condition, then the soil mass adjacent to the wall will pass into a failure state as shown in Figure 6.3. At this stage, the



**Fig. 6.2** At-rest earth pressure distribution—homogeneous soil.

**Figure 3-2 Calculating  $\beta$  with Broken Backfill Surface**

Rankine theory, as described in Section 3.6.5.2, may also be used for the design of yielding walls, for a simplified analysis (at the Structural Designer's option). The use of Rankine theory will result in a slightly more conservative design.

**3.6.5.2 Rankine Theory**

Rankine theory should be used for long-heeled cantilever walls. Refer to AASHTO LRFD Figure C3.11.5.3-1 (a) for the definition of a long heeled cantilever wall. For simplicity (at the Structural Designer's option), Rankine theory may also be used to compute lateral earth pressures on any yielding wall listed in 3.6.5.1 Coulomb Theory, although its use will result in a slightly more conservative design.

For these cases, interface friction between the wall backface and the backfill is not considered. Rankine earth pressure is applied to a plane extending vertically from the heel of the wall base, as shown in Figure 3-3.

For a horizontal backfill surface where  $\beta = 0^\circ$ , the value of the coefficient of active earth pressure (Rankine),  $K_a$ , may be taken as:

$$K_a = \tan^2 \left( 45^\circ - \frac{\phi}{2} \right)$$

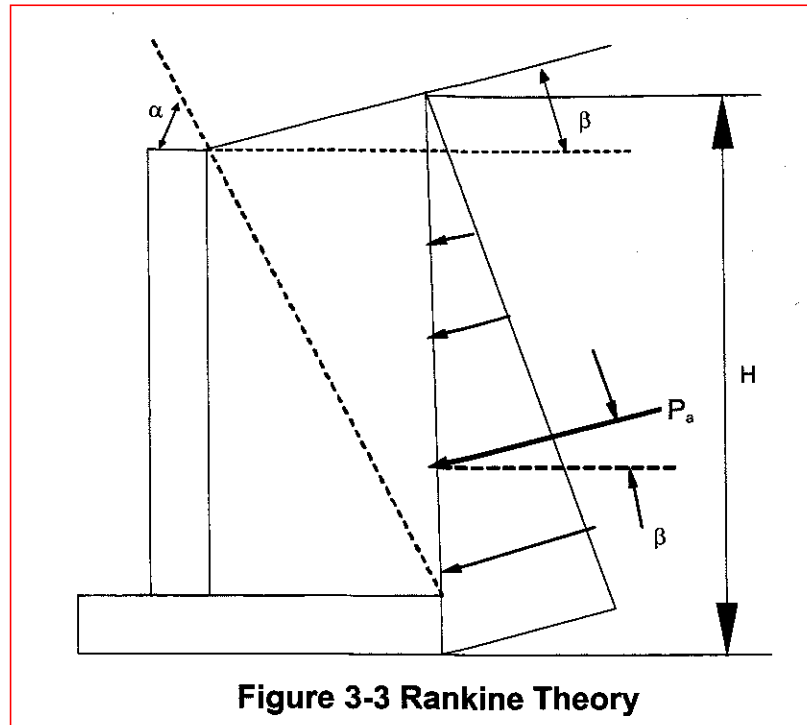
where:

$\phi$  = angle of internal soil friction (degrees), taken from Table 3-3.

$\beta$  = angle of backfill to the horizontal (degrees), as shown in Figure 3-3.

For a sloped backfill surface where  $\beta > 0^\circ$ , the coefficient of active earth pressure (Rankine),  $K_a$ , may be taken as:

$$K_a = \cos \beta \cdot \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$



**Figure 3-3 Rankine Theory**

The resultant earth pressure force,  $P_a$ , is oriented at an angle,  $\beta$ , as shown in Figure 3-3. The resultant acts at a distance,  $H/3$ , from the base of the footing.

For situations with a broken backfill surface, the active earth pressure coefficient,  $K_a$ , may be determined using a  $\beta$  value adjusted per AASHTO LRFD Figures 3.11.5.8 -1 through 3, or substituted with  $\beta^*$ , as shown in Figure 3-2.

### 3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

Values of the coefficient of passive lateral earth pressure,  $K_p$ , may be taken from Figures 3.11.5.4-1 and 2 in AASHTO LRFD or using Coulomb theory, as shown below:

$$K_p = \frac{\sin(\alpha - \phi)^2}{\sin \alpha^2 \cdot \sin(\alpha + \delta) \cdot \left( 1 - \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi + \beta)}{\sin(\alpha + \delta) \cdot \sin(\alpha + \beta)}} \right)^2}$$

where:

$\alpha$  = angle (degrees) of back of wall to the horizontal as shown in Figure 3-1.

$\phi$  = angle of internal soil friction (degrees), taken from Table 3-3.

Bearing Resistance

**Objective:**

Estimate the factored bearing resistance for a box culvert bearing on soil at the Service Limit State and Strength Limit State.

**Given:**

1. Limited lab data
2. Soil engineering properties based on correlations to SPT N-values and in-situ vane shear test results

**Assumptions:**

1. The box culvert's embedment is 2' into the streambed.
2. The proposed bearing elevation is approximately 159 feet.
3. Proposed finish roadway grade elevation is approximately 172.5 feet at the low point.
4. Proposed precast concrete box base is 20 feet wide.
5. The bottom of the box culvert will be submerged for the structure's design life.

**Estimate the factored bearing resistance at the Service Limit State:**

The use of presumptive values may be used when sufficient knowledge of geological conditions at or near the structure site exists. AASHTO LRFD 8th Edition Table C10.6.2.6.1-1 provides presumptive bearing resistances for spread footings when a settlement limited bearing resistance is appropriate. For more information see *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, p. 7.2-142.

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Medium dense to dense	2-6	2

**Recommend 2 ksf to limit settlement to 1.0 inch for Service Limit State Loads**

**2. Estimate the factored bearing resistance at the Strength Limit State:**

Foundation Width, Depth, and Water Surface

$$B := 20\text{ft}$$

$$D_f := 2.0\text{ft}$$

$$D_w := 0\text{ft}$$

$$\gamma_w := 62.4\text{pcf}$$

Total unit weight of the soil above the base slab/soil envelope

$$\gamma_{\text{above}} := 125 \cdot \text{pcf}$$

MaineDOT Bridge Design Guide p. 3-3 Soil Type 4

$$\gamma_{\text{above\_sat}} := 135 \cdot \text{pcf}$$

Foundation soils:

Foundation soils properties based on BB-WGCS-101 and -102

$$\gamma_{1d} := 93 \cdot \text{pcf}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:  
Soft Clay

$$w_{\text{sat}} := 40.6\%$$

$$\gamma_{1\text{sat}} := \gamma_{1d} \cdot (1 + w_{\text{sat}})$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:  
Table 3.1 Unit weight relationships

$$\gamma_{1\text{sat}} = 130.8 \cdot \text{pcf}$$

$$\phi := 0 \cdot \text{deg}$$

$$c := 744 \text{psf}$$

Average of 6 vane tests in BB-WGCS-101 and  
BB-WGCS-102

### **Nominal Bearing Resistance for Strength Limit States**

Reference: Munfakh, et al (2001) LRFD Article 10.6.3.1.2a

Bearing Capacity Factors (Ref. LRFD Table 10.6.3.1.2a-1)

$$N_c := 5.14$$

$$N_q := 1$$

$$N_\gamma := 0$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

$$L := 74 \cdot \text{ft}$$

$$s_c := 1 + \left( \frac{B}{L} \right) \cdot \left( \frac{N_q}{N_c} \right)$$

$$s_\gamma := 1 - 0.4 \cdot \left( \frac{B}{L} \right)$$

$$s_q := 1 + \frac{B}{L} \cdot \tan(\phi)$$

$$s_c = 1.1 \quad s_\gamma = 0.9 \quad s_q = 1$$

#### Groundwater Coefficients - LRFD Table 10.6.3.1.2a-2

The highest anticipated groundwater level should be used in design.

Assume groundwater, or stream elevation, will be above the invert of the structure for the entire design life.

$$C_{wq} := .5 \quad C_{w\gamma} := 0.5$$

#### Load Inclination factors

No knowledge of vertical and horizontal loads at this time. Use 1.0

$$i_c := 1.0 \quad i_\gamma := 1.0 \quad i_q := 1.0$$

Depth correction factors - only used when soils above the footing bearing elevation are as competent as the soils beneath the footing level. Otherwise 1.0

#### LRFD Table 10.6.3.1.2a-4

$$\frac{D_f}{B} = 0.1$$

Therefore :

$$d_q := 1.0$$

#### Terms

$$N_{cm} := N_c \cdot s_c \cdot i_c$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \cdot i_\gamma$$

$$N_{cm} = 5.4$$

$$N_{\gamma m} = 0$$

$$N_{qm} = 1$$

#### Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[ c \cdot N_{cm} + \gamma_{\text{above\_sat}} \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{1 \text{ sat}} \cdot \overline{(B \cdot N_{\gamma m})} \cdot C_{w\gamma} \right]$$

$$q_n = 4.2 \cdot \text{ksf}$$

West Gardiner, Gosline Bridge  
23090.00

Bearing Resistance  
Precast Box Culvert

Calculation by J. Manahan  
June 2021  
Checked by: LK 10/1/2021

Factored Bearing Resistance

$$\phi_b := 0.45$$

$$q_r := q_n \cdot \phi_b$$

$$q_r = 1.9 \cdot \text{ksf}$$

**Recommend a factored bearing resistance of 2 ksf for box bottom slabs 20 ft or greater.**

### 3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

### 3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

### 3.6 Earth Loads

#### 3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

**Table 3-3 Material Classification**

Soil Type	Soil Description	Internal Angle of Friction of Soil, $\phi$	Soil Total Unit Weight (pcf)	Coeff. of Friction, $\tan \delta$ , Concrete to Soil	Interface Friction, Angle, Concrete to Soil $\delta$
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29°*	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

\* The value given for the internal angle of friction ( $\phi$ ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

### 3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for  $\gamma$ ,  $\gamma_d$ , and  $\gamma_{\text{sat}}$  are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

**Table 3.1** Various Forms of Relationships for  $\gamma$ ,  $\gamma_d$ , and  $\gamma_{\text{sat}}$

Moist unit weight ( $\gamma$ )		Dry unit weight ( $\gamma_d$ )		Saturated unit weight ( $\gamma_{\text{sat}}$ )	
Given	Relationship	Given	Relationship	Given	Relationship
$w, G_s, e$	$\frac{(1+w)G_s\gamma_w}{1+e}$	$\gamma, w$	$\frac{\gamma}{1+w}$	$G_s, e$	$\frac{(G_s+e)\gamma_w}{1+e}$
$S, G_s, e$	$\frac{(G_s+Se)\gamma_w}{1+e}$	$G_s, e$	$\frac{G_s\gamma_w}{1+e}$	$G_s, n$	$[(1-n)G_s+n]\gamma_w$
$w, G_s, S$	$\frac{(1+w)G_s\gamma_w}{1+\frac{wG_s}{S}}$	$G_s, n$	$G_s\gamma_w(1-n)$	$G_s, w_{\text{sat}}$	$\left(\frac{1+w_{\text{sat}}}{1+w_{\text{sat}}G_s}\right)G_s\gamma_w$
$w, G_s, n$	$G_s\gamma_w(1-n)(1+w)$	$G_s, w, S$	$\frac{G_s\gamma_w}{1+\left(\frac{wG_s}{S}\right)}$	$e, w_{\text{sat}}$	$\left(\frac{e}{w_{\text{sat}}}\right)\left(\frac{1+w_{\text{sat}}}{1+e}\right)\gamma_w$
$S, G_s, n$	$G_s\gamma_w(1-n)+nS\gamma_w$	$e, w, S$	$\frac{eS\gamma_w}{(1+e)w}$	$n, w_{\text{sat}}$	$n\left(\frac{1+w_{\text{sat}}}{w_{\text{sat}}}\right)\gamma_w$
		$\gamma_{\text{sat}}, e$	$\gamma_{\text{sat}} - \frac{e\gamma_w}{1+e}$	$\gamma_d, e$	$\gamma_d + \left(\frac{e}{1+e}\right)\gamma_w$
		$\gamma_{\text{sat}}, n$	$\gamma_{\text{sat}} - n\gamma_w$	$\gamma_d, n$	$\gamma_d + n\gamma_w$
		$\gamma_{\text{sat}}, G_s$	$\frac{(\gamma_{\text{sat}} - \gamma_w)G_s}{(G_s - 1)}$	$\gamma_d, S$	$\left(1 - \frac{1}{G_s}\right)\gamma_d + \gamma_w$
				$\gamma_d, w_{\text{sat}}$	$\gamma_d(1 + w_{\text{sat}})$

**Table 3.2** Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, $e$	Natural moisture content in a saturated state (%)	Dry unit weight, $\gamma_d$	
			lb/ft <sup>3</sup>	kN/m <sup>3</sup>
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

**Table C10.6.2.5.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)**

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	12–20	14
	Medium dense to dense	8–14	10
	Loose	4–12	6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense	8–12	8
	Medium dense to dense	4–8	6
	Loose	2–6	3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense	6–12	8
	Medium dense to dense	2–6	4
	Loose	1–2	1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	4–8	6
	Medium stiff to stiff	2–6	3
	Soft	1–2	1

#### 10.6.2.5.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as  $0.3f'_C$ .

## 10.5.5.2.2—Spread Footings

## C10.5.5.2.2

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site-specific considerations in Article 10.5.5.2.

**Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State**

Method/Soil/Condition			Resistance Factor
Bearing Resistance	$\phi_b$	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>CPT</i>	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using <i>SPT</i>	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	$\phi_\tau$	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	$\phi_{ep}$	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

The resistance factor for sliding of cast-in-place concrete on sand is slightly lower than the other sliding resistance factors based on reliability theory analysis (Barker et al., 1991). The higher interface friction coefficient used for sliding of cast-in-place concrete on sand relative to that used for precast concrete on sand causes the cast-in-place concrete sliding analysis to be less conservative, resulting in the need for the lower resistance factor. A more detailed explanation of the development of the resistance factors provided in Table 10.5.5.2.2-1 is provided in Allen (2005).

The resistance factors for plate load tests and passive resistance were based on engineering judgment and past ASD practice.

## 10.5.5.2.3—Driven Piles

## C10.5.5.2.3

Resistance factors shall be selected from Table 10.5.5.2.3-1 based on the method used for determining the driving criterion necessary to achieve the required nominal pile bearing resistance.

Regarding load tests, and dynamic tests with signal matching, the number of tests to be conducted to justify the design resistance factors selected should be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A

Where nominal pile bearing resistance is determined by static load test, dynamic testing, wave equation, or dynamic formulas, the uncertainty in the nominal resistance is strictly due to the reliability of the resistance determination method used in the field during pile installation.

In most cases, the nominal bearing resistance of each production pile is field-verified based on compliance with a driving criterion developed using a dynamic method

Consideration should be given to the relative change in the computed nominal resistance based on effective versus gross footing dimensions for the size of footings typically used for bridges. Judgment should be used in deciding whether the use of gross footing dimensions for computing nominal bearing resistance at the strength limit state would result in a conservative design.

### 10.6.3.1.2—Theoretical Estimation

#### 10.6.3.1.2a—Basic Formulation

#### C10.6.3.1.2a

The nominal bearing resistance shall be estimated using accepted soil mechanics theories and should be based on measured soil parameters. The soil parameters used in the analyses shall be representative of the soil shear strength under the considered loading and subsurface conditions.

The nominal bearing resistance of spread footings on cohesionless soils shall be evaluated using effective stress analyses and drained soil strength parameters.

The nominal bearing resistance of spread footings on cohesive soils shall be evaluated for total stress analyses and undrained soil strength parameters. In cases where the cohesive soils may soften and lose strength with time, the bearing resistance of these soils shall also be evaluated for permanent loading conditions using effective stress analyses and drained soil strength parameters.

For spread footings bearing on compacted soils, the nominal bearing resistance shall be evaluated using the more critical of either total or effective stress analyses.

Except as noted below, the nominal bearing resistance of a soil layer, in ksf, should be taken as:

$$q_n = cN_{cn} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{\gamma m} C_{w\gamma} \quad (10.6.3.1.2a-1)$$

in which:

$$N_{cm} = N_c s_c i_c \quad (10.6.3.1.2a-2)$$

$$N_{qm} = N_q s_q d_q i_q \quad (10.6.3.1.2a-3)$$

$$N_{\gamma m} = N_{\gamma} s_{\gamma} i_{\gamma} \quad (10.6.3.1.2a-4)$$

where:

- $c$  = cohesion, taken as undrained shear strength (ksf)
- $N_c$  = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
- $N_q$  = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

The bearing resistance formulation provided in Eqs. 10.6.3.1.2a-1 through 10.6.3.1.2a-4 is the complete formulation as described in the Munfakh et al. (2001). However, in practice, not all of the factors included in these equations have been routinely used.

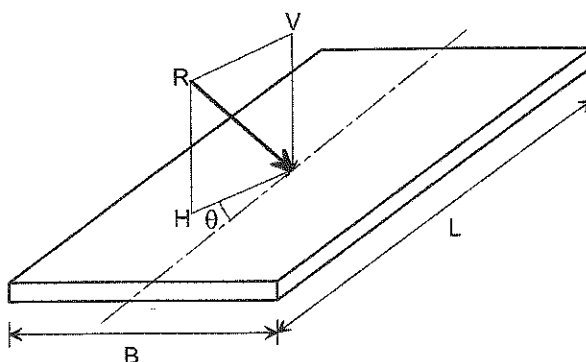


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

Table 10.6.3.1.2a-1—Bearing Capacity Factors  $N_c$  (Prandtl, 1921),  $N_q$  (Reissner, 1924), and  $N_\gamma$  (Vesic, 1975)

$\phi_f$	$N_c$	$N_q$	$N_\gamma$	$\phi_f$	$N_c$	$N_q$	$N_\gamma$
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Table 10.6.3.1.2a-2—Coefficients  $C_{wq}$  and  $C_{w\gamma}$  for Various Groundwater Depths

$D_w$	$C_{wq}$	$C_{w\gamma}$
0.0	0.5	0.5
$D_f$	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

Table 10.6.3.1.2a-3—Shape Correction Factors  $s_c$ ,  $s_\gamma$ ,  $s_q$ 

Factor	Friction Angle	Cohesion Term ( $s_c$ )	Unit Weight Term ( $s_\gamma$ )	Surcharge Term ( $s_q$ )
Shape Factors $s_c, s_\gamma, s_q$	$\phi_f = 0$	$1 + \left( \frac{B}{5L} \right)$	1.0	1.0
	$\phi_f > 0$	$1 + \left( \frac{B}{L} \right) \left( \frac{N_q}{N_c} \right)$	$1 - 0.4 \left( \frac{B}{L} \right)$	$1 + \left( \frac{B}{L} \tan \phi_f \right)$

$$d_q = 1 + 2 \tan \phi_f (1 - \sin \phi_f)^2 \arctan \left( \frac{D_f}{B} \right) \quad (10.6.3.1.2a-10)$$

Eq. 10.6.3.1.2a-10 has been verified to cover a range of friction angle,  $\phi_f$ , of 32 degrees to 42 degrees, and a range of  $D_f/B$  of 1 to 8. Depth correction factor values beyond this range have not been verified at this time.

where:

$d_q$  = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation(dim)

$\phi_f$  = angle of internal friction of soil (degrees)

$D_f$  = footing embedment depth (ft)

$B$  = footing width (ft)

Arctan ( $D_f/B$ ) is in radians.

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0. The depth correction factor,  $d_q$ , shall not exceed 1.4.

#### 10.6.3.1.2b—Considerations for Punching Shear

#### C10.6.3.1.2b

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters  $c^*$  and  $\phi^*$  in Eqs. 10.6.3.1.2b-1 and 10.6.3.1.2b-2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \quad (10.6.3.1.2b-1)$$

$$\phi^* = \tan^{-1}(0.67 \tan \phi_f) \quad (10.6.3.1.2b-2)$$

where:

$c^*$  = reduced effective stress soil cohesion for punching shear (ksf)

$\phi^*$  = reduced effective stress soil friction angle for punching shear (degrees)

Local shear failure is characterized by a failure surface that is similar to that of a general shear failure but that does not extend to the ground surface, ending somewhere in the soil below the footing. Local shear failure is accompanied by vertical compression of soil below the footing and visible bulging of soil adjacent to the footing but not by sudden rotation or tilting of the footing. Local shear failure is a transitional condition between general and punching shear failure. Punching shear failure is characterized by vertical shear around the perimeter of the footing and is accompanied by a vertical movement of the footing and compression of the soil immediately below the footing but does not affect the soil outside the loaded area. Punching shear failure occurs in loose or compressible soils, in weak soils under slow (drained) loading, and in dense sands for deep footings subjected to high loads.

Modulus of Subgrade Reaction

**Objective:**

Estimate the modulus of subgrade reaction for the box culvert base slab design.

**Given:**

1. Limited lab data, SPT N-values, and in-situ vane shear test results.

**Assumptions:**

1. The proposed bearing elevation of base slab is approximately 159 feet.
2. Proposed finished roadway grade is approximately 172.5.
3. Proposed precast concrete box is 20 feet wide and approximately 74 feet long.
4. The subsurface conditions present at the proposed bearing elevation is medium stiff glaciomarine clayey silt and silty clay, with  $S_u=491-1205$  psf,  $S_u(\text{average})=744$  psf.
5. The bottom of the box culvert will be submerged for the structure's design life.
6.  $q_u/2 = s_u$   $q_u = 744 \times 2$  psf = 1488 psf = 0.74 TSF

**Published values of subgrade modulus**

Published values of subgrade modulus in clayey silt:

Bowles Foundation Analysis and Design, 5th ed. Table 9-1:

Range of modulus of subgrade reaction 44 to 88 pci

Subgrade of clayey soil  $q_u < 200$  kPa = 4200 psf, lower limit:  $k_s = 44$  pci

FHWA Geotechnical Engineering Circular (GEC) No. 6, Figure 8-3:

Fine grained soils  $q_u = 0.74$  TSF, use lowest value presented on Fine Grained Curve

$K_{v1}$ , 23 pci / 2 = 11.5 pci

Das Principles of Foundation Engineering, 7th ed. Table 6.2:

Typical subgrade reaction values for 0.3 m x 0.3 m plate

No value for medium stiff clay, use stiff clay, 37 - 92 pci:  $k_{0.3} (k_1) = 65$  pci

### Adjust Published values for dimensions of base slab

Published range for medium stiff, silty clay subgrade is 11.5 - 65 pci

Assume a subgrade modulus of 40.2 pci, average of 11.5, 44, and 65 pci.

Value of  $k_{s1} = 40.2$  pci is for a 1 ft x 1 ft plate. Adjust to the dimensions of the box culvert base (Width B = 20 ft, Length L = 74 ft).

Square to rectangle base adjustment:

$$k_{s1} := 40.2 \text{ pci} \quad B := 20 \text{ ft} \quad L := 74 \text{ ft}$$

$$k := \frac{k_{s1} \cdot \left[ 1 + 0.5 \left( \frac{B}{L} \right) \right]}{1.5}$$

Das, Principles of Foundation  
Engineering 7th Ed. P. 311 Eqn. 6.44

$$k = 30.4 \cdot \text{pci}$$

Recommend a subgrade modulus of 30.4 pci

for either a horizontal or lateral modulus of subgrade reaction is

$$k_s = A_s + B_s Z^n \quad (9-10)$$

for either horizontal or vertical members

for depth variation

interest below ground

to give  $k_s$  the best fit (if load test or other data are available)

variation may be zero; at the ground surface  $A_s$  is zero for a lateral  $k_s$

$> 0$ . For footings and mats (plates in general),  $A_s > 0$  and  $B_s \approx 0$ .

used with the proper interpretation of the bearing-capacity equations (with  $d_i$  factors dropped) to give

$$q_{ult} = cN_{cs} + \gamma ZN_{qs} + 0.5\gamma BN_{\gamma s} \quad (9-10a)$$

$$cN_{cs} + 0.5\gamma BN_{\gamma s}) \quad \text{and} \quad B_s Z^1 = C(\gamma N_{qs})Z^1$$

to estimate  $k_s$ . In these equations the Terzaghi or Hansen bearing-

capacity factors are used. The  $C$  factor is 40 for SI units and 12 for Fps, using the same

values at a 0.0254-m and 1-in. settlement but with no SF, since this equation

assumes there is concern that  $k_s$  does not increase without bound with

depth. The  $B_s Z$  term by one of two simple methods:

$$\text{Method 1: } B_s \tan^{-1} \frac{Z}{D}$$

$$\text{Method 2: } \frac{B_s}{D^n} Z^n = B'_s Z^n$$

depth of interest, say, the length of a pile

depth of interest

estimate of the exponent

to estimate a value of  $k_s$  to determine the correct order of magnitude

obtained using one of the approximations given here. Obviously if a

value is three times larger than the table range indicates, the computations

will have a possible gross error. Note, however, if you use a reduced value of

(or 12 mm) instead of 0.0254 m you may well exceed the table range.

If a computational error (or a poor assumption) is found then use judgment

to select table values are intended as guides. The reader should not use, say,

even as a "good" estimate.

shown in Fig. 9-9c (and used in your diskette program FADBEMLP as

estimated at some small value of, say, 6 to 25 mm, or from inspection

of a load test was done. It might also be estimated from a triaxial

test "ultimate" or at the maximum pressure from the stress-strain plot.

compute

$$X_{\max} = \epsilon_{\max}(1.5 \text{ to } 2B)$$

TABLE 9-1

Range of modulus of subgrade

reaction  $k_s$

Use values as guide and for comparison when using approximate equations  $\frac{kN}{M^3} \rightarrow \frac{lb}{in^3} : \frac{224.8 lb}{1 kN} * \frac{1 M^3}{61023.7 in^3} = .003684 \frac{kN}{M^3} = 1 \frac{lb}{in^3}$

Soil	$k_s$ , kN/m <sup>3</sup>	$k_s$ , lb/in <sup>3</sup>
Loose sand	4800-16,000	18 - 59
Medium dense sand	9600-80,000	35 - 295
Dense sand	64,000-128,000	236 - 472
Clayey medium dense sand	32,000-80,000	118 - 295
Silty medium dense sand	24,000-48,000	88 - 177
Clayey soil:		
$q_a \leq 200$ kPa	12,000-24,000	44 - 88
$200 < q_a \leq 800$ kPa	24,000-48,000	88 - 177
$q_a > 800$ kPa	$> 48,000$	$> 177$

44 pci

The 1.5 to 2B dimension is an approximation of the depth of significant stress-strain influence (Boussinesq theory) for the structural member. The structural member may be either a footing or a pile.

**Example 9-5.** Estimate the modulus of subgrade reaction  $k_s$  for the following design parameters:

$$B = 1.22 \text{ m} \quad L = 1.83 \text{ m} \quad D = 0.610 \text{ m}$$

$$q_a = 200 \text{ kPa (clayey sand approximately 10 m deep)}$$

$$E_s = 11.72 \text{ MPa (average in depth } 5B \text{ below base)}$$

**Solution.** Estimate Poisson's ratio  $\mu = 0.30$  so that

$$E'_s = \frac{1 - \mu^2}{E_s} = \frac{1 - 0.3^2}{11.72} = 0.07765 \text{ m}^2/\text{MN}$$

For center:

$$H/B' = 5B/(B/2) = 10 \text{ (taking } H = 5B \text{ as recommended in Chap. 5)}$$

$$L/B = 1.83/1.22 = 1.5$$

From these we may write

$$I_s = 0.584 + \frac{1 - 2(0.3)}{1 - 0.3} (0.023) = 0.597$$

using Eq. (5-16) and Table 5-2 (or your program FFACTOR) for factors 0.584 and 0.023.

At  $D/B = 0.61/1.22 = 0.5$ , we obtain  $I_F = 0.80$  from Fig. 5-7 (or when using FFACTOR for the  $I_s$  factors). Substitution into Eq. (9-7) with  $B' = 1.22/2 = 0.61$ , and  $m = 4$  yields

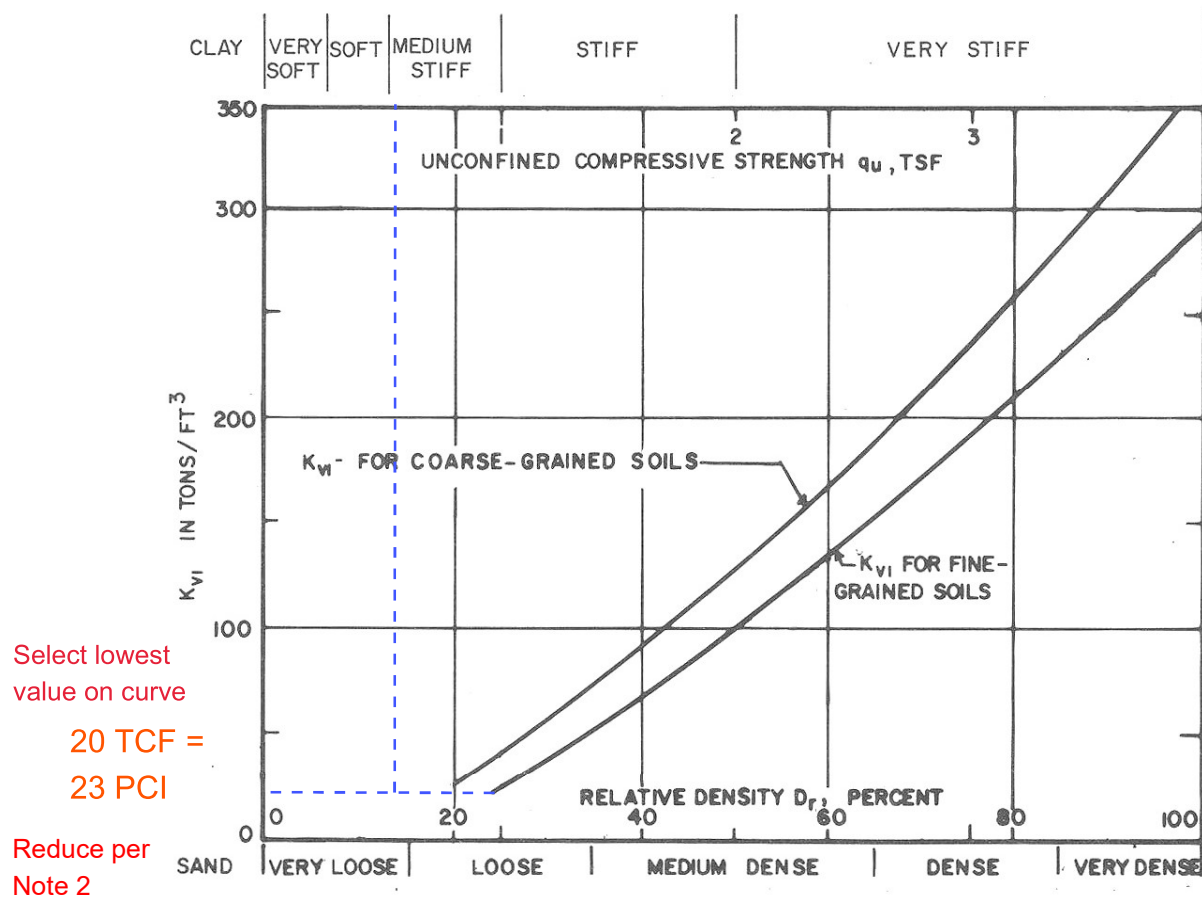
$$k_s = \frac{1}{0.61(0.07765)(4 \times 0.597)(0.8)} = 11.05 \text{ MN/m}^3$$

You should note that  $k_s$  does not depend on the contact pressure of the base  $q_o$ .

For corner:

$$H/B' = 5B/B = 5(1.22)/1.22 = 5$$

[from Table 5-2 with  $L/B = 1.5$  obtained for Eq. (5-16)]



#### DEFINITIONS

$\Delta H_i$  = IMMEDIATE SETTLEMENT OF FOOTING  
 $q$  = FOOTING UNIT LOAD IN tsf  
 $B$  = FOOTING WIDTH

$D$  = DEPTH OF FOOTING BELOW GROUND SURFACE

$K_{vi}$  = MODULUS OF VERTICAL SUBGRADE REACTION

$$\frac{\text{ton}}{\text{ft}^3} \rightarrow \frac{\text{lb}}{\text{in}^3} = \frac{2000 \text{ lb}}{1 \text{ ton}} \cdot \frac{1 \text{ ft}^3}{1728 \text{ in}^3} = 1.157 \frac{\text{ton}}{\text{ft}^3} \rightarrow 1 \frac{\text{lb}}{\text{in}^3}$$

#### COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)  
 SHALLOW FOOTINGS  $D \leq B$

FOR  $B \leq 20 \text{ FT}$ :

$$\Delta H_i = \frac{4 q B^2}{K_{vi} (B+1)^2}$$

FOR  $B \geq 40 \text{ FT}$ :

$$\Delta H_i = \frac{2 q B^2}{K_{vi} (B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF  $B$

DEEP FOUNDATION  $D \geq 5B$

FOR  $B \leq 20 \text{ FT}$ :

$$\Delta H_i = \frac{2 q B^2}{K_{vi} (B+1)^2}$$

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.

2. VALUES OF  $K_{vi}$  SHOWN FOR COARSE-GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUNDWATER LEVEL AT A DEPTH OF AT LEAST  $1.5B$  BELOW BASE OF FOOTING. IF GROUNDWATER IS AT BASE OF FOOTING, USE  $K_{vi}/2$  IN COMPUTING SETTLEMENT

Figure 8-3: Modulus of Subgrade Reaction (NAVFAC, 1986a)

Equation (6.44) indicates that the value of  $k$  for a very long foundation with a width  $B$  is approximately  $0.67k_{(B \times B)}$ .

The modulus of elasticity of granular soils increases with depth. Because the settlement of a foundation depends on the modulus of elasticity, the value of  $k$  increases with the depth of the foundation.

Table 6.2 provides typical ranges of values for the coefficient of subgrade reaction,  $k_{0.3}(k_1)$ , for sandy and clayey soils.

For long beams, Vesic (1961) proposed an equation for estimating subgrade reaction, namely,

$$k' = Bk = 0.65 \sqrt[12]{\frac{E_s B^4}{E_F I_F}} \frac{E_s}{1 - \mu_s^2}$$

or

$$k = 0.65 \sqrt[12]{\frac{E_s B^4}{E_F I_F}} \frac{E_s}{B(1 - \mu_s^2)} \quad (6.45)$$

where

$E_s$  = modulus of elasticity of soil

$B$  = foundation width

$E_F$  = modulus of elasticity of foundation material

$I_F$  = moment of inertia of the cross section of the foundation

$\mu_s$  = Poisson's ratio of soil

$$\frac{MN}{m^3} \rightarrow \frac{lb}{in^3} : \frac{224809 lb}{1 MN} * \frac{1 m^3}{61024 in^3} \rightarrow 3.684 \frac{lb}{in^3} = \frac{1 MN}{M^3}$$

**Table 6.2** Typical Subgrade Reaction Values,  $k_{0.3}(k_1)$

Soil type	$k_{0.3}(k_1)$ MN/m <sup>3</sup>	pci
Dry or moist sand:		
Loose	8–25	29 - 92
Medium	25–125	92 - 461
Dense	125–375	461 - 1382
Saturated sand:		
Loose	10–15	37 - 55
Medium	35–40	129 - 147
Dense	130–150	478 - 553
Clay:		
Stiff	10–25	37 - 92
Very stiff	25–50	92 - 184
Hard	>50	> 184

The unit of  $k$  is  $\text{kN/m}^3$ . The value of the coefficient of subgrade reaction is not a constant for a given soil, but rather depends on several factors, such as the length  $L$  and width  $B$  of the foundation and also the depth of embedment of the foundation. A comprehensive study by Terzaghi (1955) of the parameters affecting the coefficient of subgrade reaction indicated that the value of the coefficient decreases with the width of the foundation. In the field, load tests can be carried out by means of square plates measuring  $0.3 \text{ m} \times 0.3 \text{ m}$ , and values of  $k$  can be calculated. The value of  $k$  can be related to large foundations measuring  $B \times B$  in the following ways:

### **Foundations on Sandy Soils**

For foundations on sandy soils,

$$k = k_{0.3} \left( \frac{B + 0.3}{2B} \right)^2 \quad (6.42)$$

where  $k_{0.3}$  and  $k$  = coefficients of subgrade reaction of foundations measuring  $0.3 \text{ m} \times 0.3 \text{ m}$  and  $B \text{ (m)} \times B \text{ (m)}$ , respectively (unit is  $\text{kN/m}^3$ ).

### **Foundations on Clays**

For foundations on clays,

$$k (\text{kN/m}^3) = k_{0.3} (\text{kN/m}^3) \left[ \frac{0.3 \text{ (m)}}{B \text{ (m)}} \right] \quad (6.43)$$

The definitions of  $k$  and  $k_{0.3}$  in Eq. (6.43) are the same as in Eq. (6.42).

For rectangular foundations having dimensions of  $B \times L$  (for similar soil and  $q$ ),

$$k = \frac{k_{(B \times B)} \left( 1 + 0.5 \frac{B}{L} \right)}{1.5} \quad (6.44)$$

Method 1:

where

$k$  = coefficient of subgrade modulus of the rectangular foundation ( $L \times B$ )  
 $k_{(B \times B)}$  = coefficient of subgrade modulus of a square foundation having dimension of  $B \times B$

Settlement

## Settle3D Analysis Information

### West Gardiner Gosline Bridge

#### Project Settings

Document Name	23090 West Gardiner Settlement-r2.s3z
Project Title	West Gardiner Gosline Bridge
Analysis	Immediate and consolidation settlement
Author	Manahan, -r2 LK 9/28/2021
Company	MaineDOT
Date Created	9/7/2021

#### Comments

SVI Load = 1.25 ksf  
Delta q = 1.25 psf at bottom slab elevation outside horizontal limits of existing pipe arch  
Model 2-foot crushed stone mat with geogrid  
Stress Computation Method Boussinesq  
Time-dependent Consolidation Analysis  
Time Units years  
Permeability Units feet/year  
Calculate settlement with mean stress  
Use average properties to calculate layered stresses

#### Stage Settings

Stage #	Name	Time [years]
1	Immediate	0
2	Consolidation	1
3	Long-term	50

#### Results

Time taken to compute: 0.0577439 seconds

#### Stage: Immediate = 0 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	1.76766
Consolidation Settlement [in]	0	0.101725
Immediate Settlement [in]	0	1.66593
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	1.25
Effective Stress [ksf]	0	2.37566
Mean Stress [ksf]	0	0.9375
Total Stress [ksf]	0	3.89376
Total Strain	0	0.02666
Pore Water Pressure [ksf]	0	1.5932
Excess Pore Water Pressure [ksf]	0	1.25
Degree of Consolidation [%]	0	43.0898
Pre-consolidation Stress [ksf]	0.0008625	2.98667
Over-consolidation Ratio	1	1.42728
Void Ratio	0	0.949998
Permeability [ft/y]	0	0.145079
Coefficient of Consolidation [ft <sup>2</sup> /y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.566179

#### Stage: Consolidation = 1 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	2.39089
Consolidation Settlement [in]	0	0.724954
Immediate Settlement [in]	0	1.66593
Secondary Settlement [in]	0	0
Loading Stress [ksf]	0	1.25
Effective Stress [ksf]	0	2.98214
Mean Stress [ksf]	0	0.9375
Total Stress [ksf]	0	3.89376
Total Strain	0	0.0435939
Pore Water Pressure [ksf]	0	0.911619
Excess Pore Water Pressure [ksf]	0	0.0380194
Degree of Consolidation [%]	0	97.2645
Pre-consolidation Stress [ksf]	0.0008625	2.98667
Over-consolidation Ratio	1	1.01431
Void Ratio	0	0.943294
Permeability [ft/y]	0	0.145079
Coefficient of Consolidation [ft <sup>2</sup> /y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	97.0425
Undrained Shear Strength	0	0.595673

### Stage: Long-term = 50 y

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.33822
Consolidation Settlement [in]	0	0.769226
Immediate Settlement [in]	0	1.66593
Secondary Settlement [in]	0	0.903063
Loading Stress [ksf]	0	1.25
Effective Stress [ksf]	0	3.02016
Mean Stress [ksf]	0	0.9375
Total Stress [ksf]	0	3.89376
Total Strain	0	0.057392
Pore Water Pressure [ksf]	0	0.8736
Excess Pore Water Pressure [ksf]	-2.86168e-016	5.71641e-016
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.0008625	3.01836
Over-consolidation Ratio	1	1.00138
Void Ratio	0	0.915913
Permeability [ft/y]	0	0.145079
Coefficient of Consolidation [ft <sup>2</sup> /y]	0	36.5
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	100
Undrained Shear Strength	0	0.597203

## Loads

### 1. Rectangular Load

Length	52 ft
Width	20 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1040 ft <sup>2</sup>
Load	1.25 ksf
Depth	13 ft
Installation Stage	Immediate = 0 y

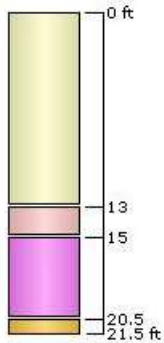
### Coordinates

X [ft]	Y [ft]
4.61853e-014	-1.42109e-014
52	-1.42109e-014
52	20
4.61853e-014	20





## Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Loose Fill	13	0	No
2	2-Foot Crushed Stone Mat with Geogrid	2	13	No
3	Glaciomarine Medium Stiff Clay	5.5	15	No
4	Marine Very Dense Sand	1	20.5	No



## Soil Properties

Property	Loose Fill	Glaciomarine Medium Stiff Clay	Marine Very Dense Sand	2-Foot Crushed Stone Mat with Geogrid
Color				
Unit Weight [kips/ft <sup>3</sup> ]	0.115	0.108	0.121	0.125
Saturated Unit Weight [kips/ft <sup>3</sup> ]	0.129	0.154	0.139	0.13
Poisson's Ratio	0.25	0.2	0.25	0.2
Immediate Settlement	Enabled	Enabled	Enabled	Enabled
E [ksf]	300	42	610	2000
Eur [ksf]	1200	168	2440	8000
Primary Consolidation	Disabled	Enabled	Disabled	Disabled
Material Type		Non-Linear		
Cc		0.4		
Cr		0.04		
e0		1		
OCR	1	1.6	1	1
Cv [ft <sup>2</sup> /y]		36.5		
B-bar		1		
Secondary Consolidation	Disabled	Mesri	Disabled	Disabled
Ca/Cc		0.04		
Undrained Su A [kips/ft <sup>2</sup> ]	0	0	0	0
Undrained Su S	0.2	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8	0.8
Piezo Line ID	1	1	1	1

## Groundwater

Groundwater method Piezometric Lines

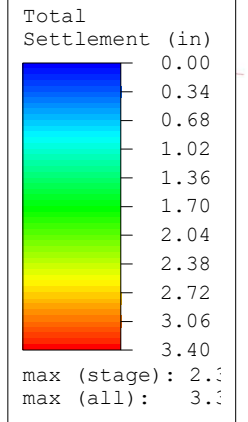
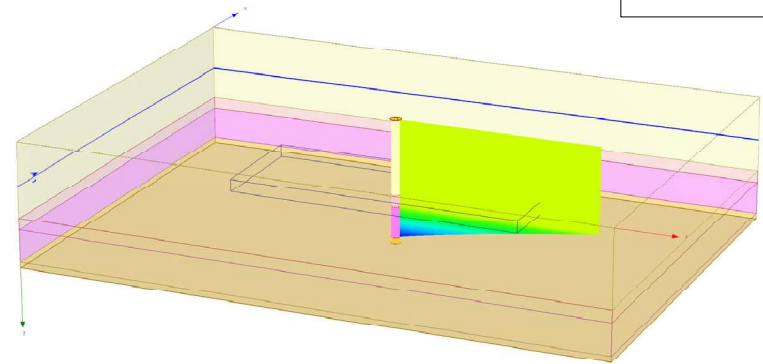
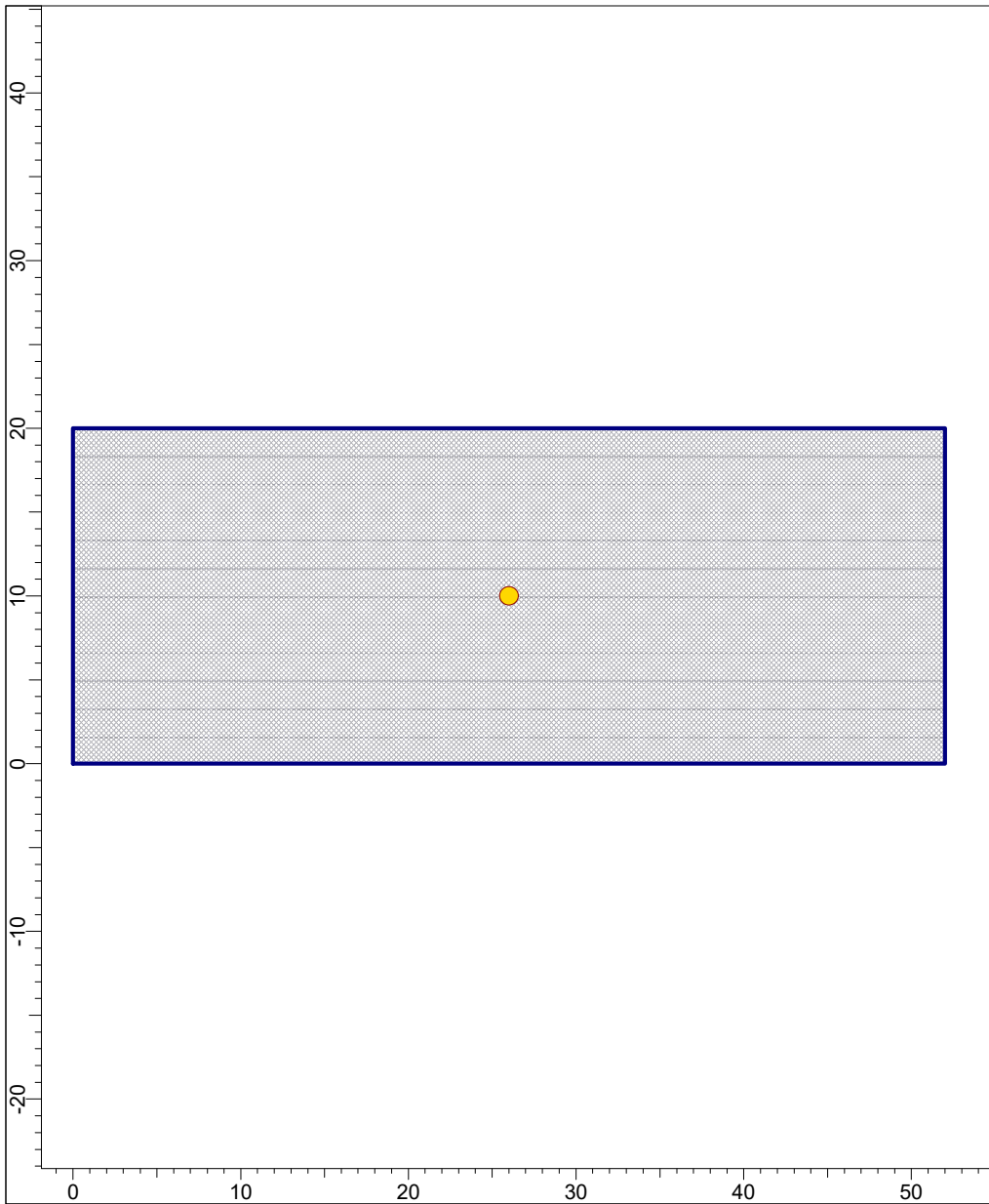
Water Unit Weight 0.0624 kips/ft<sup>3</sup>


## Piezometric Line Entities

ID	Depth (ft)
1	7.5 ft

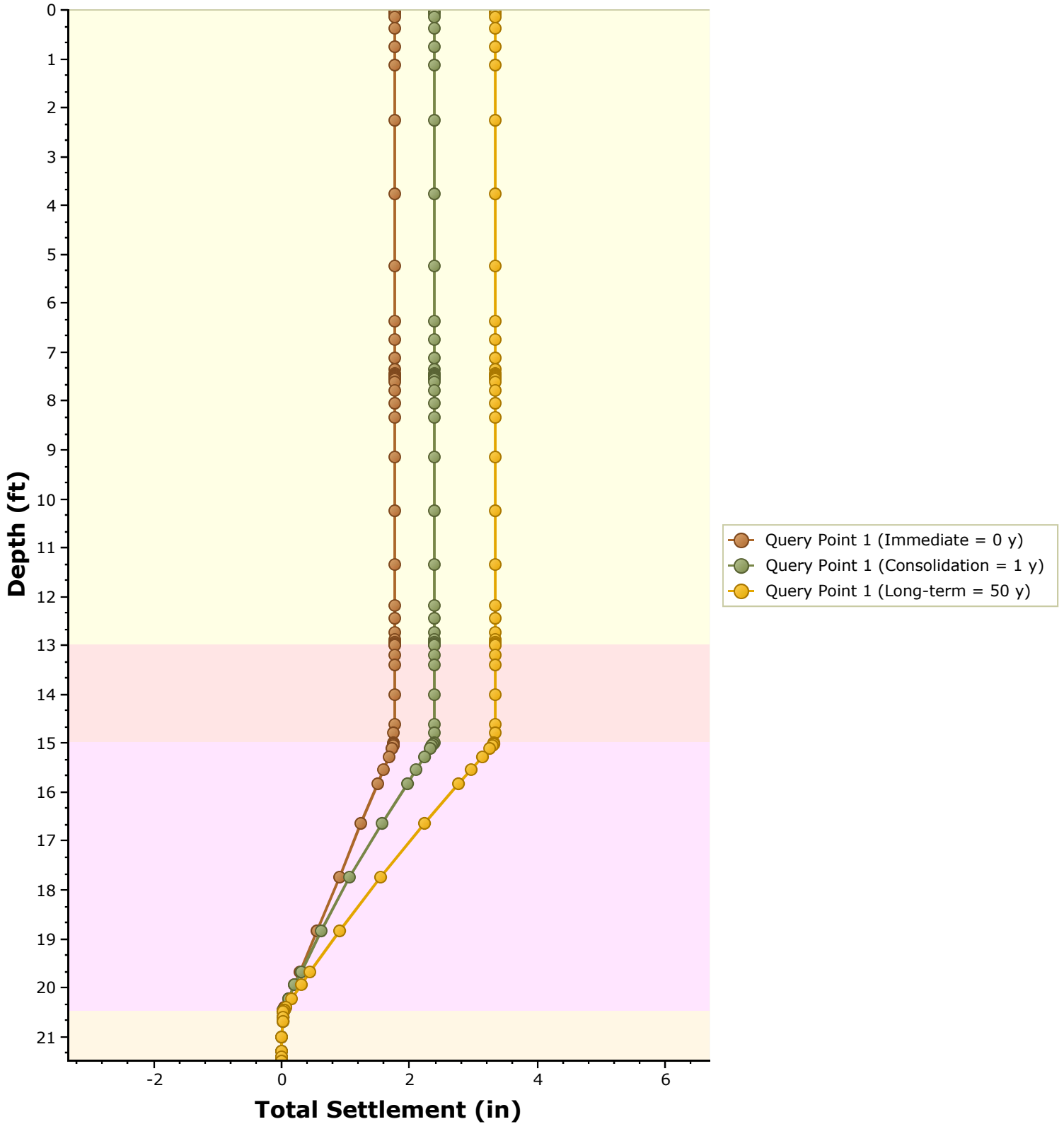
## Query Points


Point #	(X,Y) Location	Number of Divisions
1	26, 10	Auto: 67



	Project			West Gardiner Gosline Bridge	
	Analysis Description			Immediate and consolidation settlement	
	Drawn By		Manahan, -r2 LK 9/28/2021	Company	MaineDOT
	Date		9/7/2021	File Name	23090 West Gardiner Settlement-r2.s3z

# Total Settlement vs. Depth



	Project			West Gardiner Gosline Bridge
	Analysis Description			Immediate and consolidation settlement
	Drawn By		Manahan, -r2 LK 9/28/2021	Company MaineDOT
	Date		9/7/2021	File Name 23090 West Gardiner Settlement-r2.s3z

Frost

**Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table, BDG  
Section 5.2.1.**

From Design Freezing Index Map: **West Gardiner, Maine**

DFI = 1600 degree-days.

Case 1 - coarse grained soils W=15% (BB-WGCS-101 1D).

For DFI = 1600

at w=20%

$$d_1 := 70.2\text{in}$$

at w=10%

$$d_2 := 84.8\text{in}$$

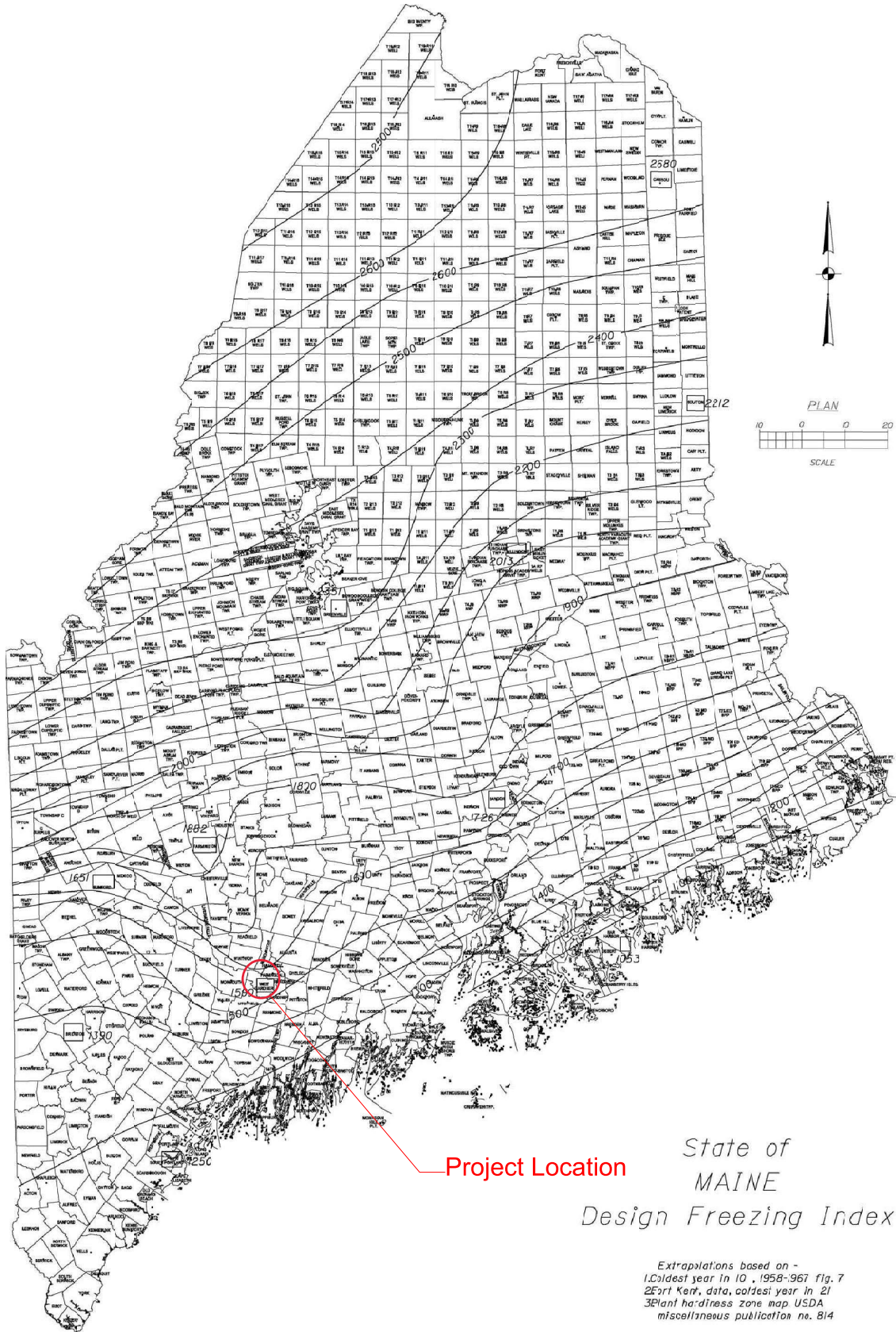
Depth of Frost Penetration

$$d := \frac{d_2 + d_1}{2}$$

$$d = 77.5\text{in}$$

$$d = 6.5\text{ft}$$

Figure 5-1 Maine Design Freezing Index Map



## 5.2 General

### MaineDOT Bridge Design Guide

#### 5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

**Table 5-1 Depth of Frost Penetration**

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0